PORE PRESSURE EFFECTS ON THE MECHANICAL PROPERTIES OF CONCRETE

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a	ratio of the area of solid contact to the gross area of the soil mass
A	pore pressure parameter
A_g	area of contact between air and soil
A_G	gross area of soil mass
A_s	area of solid to solid contact between soil grains
A_w	area of contact between water and soil
b	ratio of the area of liquid contact to the gross area of the soil mass
B	pore pressure parameter
C_c	compressibility of soil structure
C_v	compressibility of fluid (air and water) in a soil mass
f_c	confined compressive strength of concrete
f_c^{max}	maximum axial applied stress on concrete
f_c^{min}	minimum axial applied stress on concrete
f'_c	standard compressive strength of concrete
h	steady state pressure head in pore water
K	bulk modulus of dry aggregate
K_s	intrinsic bulk modulus of solid grains in aggregate
n	porosity of soil
N	number of cycles of loading to failure
P	total normal load
R	ratio of minimum applied axial stress to maximum applied axial stress

s	standard deviation
S	maximum stress level, f_c^{max}/f_c'
u_a	pore air pressure
u_w	pore water pressure
γ_w	unit weight of water
Δu_a	change in pore pressure due to change in confining pressure
Δu_d	change in pore pressure due to change in deviator stress
Δu_w	change in pore pressure
$\Delta\sigma_1$	change in major principal stress
$\Delta\sigma_3$	change in minor principal stress
σ	total stress
σ'	effective stress
σ_c	all-around confining stress
σ_g	stress in gaseous phase of a soil mass
σ_s	stress on solid soil grains
σ_w	stress in liquid phase of a soil mass
ϕ	angle of internal friction of a porous material
ψ	angle of internal friction of solid materials comprising the soil or rock

CHAPTER I

INTRODUCTION

1.1 Statement of the Problem

1.1.1 General

Concrete is gaining increasing acceptance as a structural material in place of steel for use in numerous marine and offshore structures (1). Problems may arise in assessing the strength and behavior of concrete in such new environments. In spite of a wealth of information available on properties of concrete, data regarding certain properties of concrete under particular conditions are insufficient. This is especially true in the case of the use of concrete for offshore structures. Of the several factors that affect the properties of concrete in marine environments, only the action of interstitial water pressure will be considered in this study.

1.1.2 Earlier Hydraulic Structures

The use of concrete in water environment is not new and in fact has been used for several decades in hydraulic structures such as dams, weirs and several other structures constantly submerged under water. Recognition of the interstitial water pressure as a significant factor affecting the stability of water retaining structures was slow and the influence of pore pressure on the strength of saturated and submerged concrete is yet to be evaluated. In the stability calculation of a dam the uplift caused by the interstitial water pressure was seldom accounted. However, after the 1895 failure of the Bouzey Dam in Germany, serious attention was paid to the aspect of uplift in the stability calculations of dams (2). Even so, many controversies existed as to the magnitude of uplift pressure and the area over which the said pressure acted (3-7). It was firmly believed that concrete was basically impermeable and the existence of incipient cracks was the main reason for developing pore pressures. It is now well established that for pore pressure to develop, concrete need not be assumed to be cracked or injured in any manner whatsoever (4, 5, 7, 8, and 9).

1.1.3 Pore Water Pressure

Terzaghi was instrumental in defining pore pressure in the modern sense of the term and was the first to introduce the concept of effective stress in soil mechanics (10). Later he extended the same concept to rock and concrete and held the effective stress law to be valid for both (9). Several authors have since held the effective stress law to be applicable to concrete (8, 11, 12).

There are several mechanisms by which pore pressure could develop in the pores of concrete. These pressures can either be positive or negative. Negative pore pressures usually develop in unsaturated concrete (13). Nearly all rocks as well as concrete become dilatant, prior to fracture in compression, representing an increase in porosity (14). Hence saturated concrete subjected to compression may possibly develop negative pore pressures in the dilatant state just prior to fracture. The magnitude of the pore pressure developed in a pore depends on several factors, the applied axial stress and the confining pressure, the degree of saturation and several parameters connected with pore and fluid characteristics (15). The positive pore pressures are of greatest importance since they are usually associated with possible strength reduction in concrete and cause uplift in hydraulic structures.

Terzaghi and Rendulic (16) have shown that in the case of triaxial compression tests on unjacketed concrete specimens the compressive strength of concrete remains unchanged even under very high lateral pressures. Terzaghi (9) held that for the above tests, the pore water pressure was equal to the confining pressure and the effect of confining pressure on the compressive strength of concrete was imperceptible. This is probably one of the reasons why little attention is paid to the aspect of pore pressure in concrete structures completely submerged under water and subjected to static loadings.

1.1.4 Influence of Internal Moisture Content on Strength of Concrete

It is generally recognized that a compression specimen tested in the air dry condition will exhibit significantly higher strength than that of corresponding concrete tested in a saturated condition (17, 18). The reduction in strength of the saturated concrete is associated with the development of pore pressure or sometimes crudely called as the wedge action of water in the pores (17, 18). Kaplan (19) from his experiments found that the influence of various rates of loading on the strength are more pronounced on moist concrete than on corresponding dry concrete. He explained the observed difference in behavior as possibly due to the pore pressures developed in the moist specimens during loading.

In the case of saturated concrete subjected to dynamic or fatigue loading the development of load induced pore pressure could be significant and be instrumental in the initiation of crack or propagation of crack once initiated. In view of the foregoing a knowledge of the actual behavior of saturated concrete subjected to fatigue loading is of fundamental importance for the safe design of offshore structures.

1.2 Purpose and Scope

The objectives of this study are to critically review the current state of knowledge on the effects of pore pressure on the mechanical properties of concrete and to experimentally investigate the same by means of fatigue tests on saturated concrete. In the experimental investigation the influences of the effect of submergence, the range of applied stress, rest periods, rate of loading and the size of the specimens will be studied.

CHAPTER II

LITERATURE REVIEW

The engineering properties of concrete such as strength, durability, permeability, etc. are directly influenced or controlled by the presence of pores in the cement paste and the physical charateristics of these pores. Hence a general description of the pore structure may be helpful.

2.1 Pore Structure of Concrete

Concrete is inherently porous as not all the space between aggregate particles becomes filled with solid cementing material. The void system in the concrete is due to the presence of capillary pores, gel pores, air voids, and aggregate pores.

2.1.1 Capillary Pores

The water-filled space in a freshly mixed neat cement paste represents space that is available for the formation of cement hydration products (20). Usually the amount of water used in concrete mixes is more than that required for complete hydration of cement. As hydration proceeds, the space occupied by water is continually reduced by the formation of the hydrated gel which has a bulk volume larger than the original unhydrated cement. That part of the original water space which has not become filled with hydration products constitutes the capillary system.

At higher water-cement ratios the gel volume is not sufficient to fill completely all the original water space in the paste, even after complete hydration of cement. This can result in pastes having relatively large volumes of capillaries. Verbeck (20) describes the capillary system as submicroscopic voids randomly distributed throughout the hydrated cement paste matrix and varying in diameter from about 10 nm to 10 μm depending upon the original water cement ratio and degree of hydration of cement.

The capillary pores are of negligible volume in the case of a fully hydrated cement paste with a water-cement ratio of approximately 0.36 and constitute about 30 percent of the volume of cement paste in the case of a fully hydrated cement paste with a water-cement ratio of 0.7 (18, 20). The capillary pores may or may not be directly interconnected.

2.1.2 Gel Pores

The cement gel, resulting from the hydration of portland cement, is porous, and the gel pores are interconnected interstitial spaces between gel particles (18). The size of the gel pores varies from about 10 nm to 0.5 in diameter. The gel pores occupy about 28 percent of the total volume of gel independent of the water-cement ratio of the mix (21). The gel pores are the only means of connection between the capillary pores that are not directly interconnected. The distribution of the gel and capillary pores is diagrammatically represented in Figure 1. The solid dots represent gel particles, interstitial spaces are gel pores, and spaces such as those marked C are capillary pores. The size of gel pores is exaggerated.

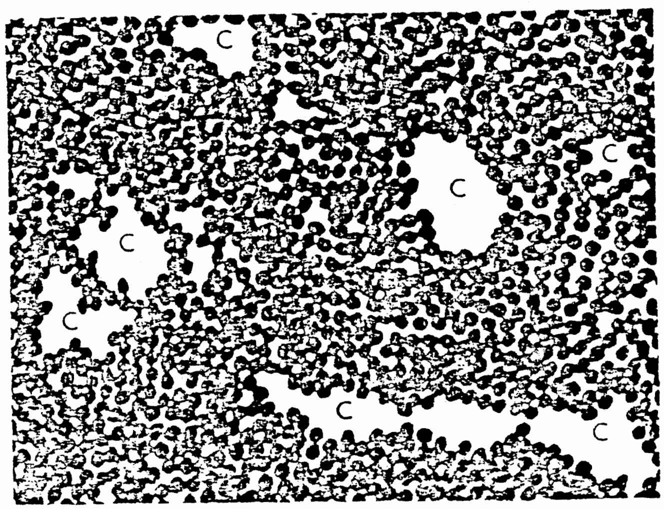


Figure 1. Simplified Model of
Paste Structure
(18, 22)

2.1.3 Air Voids

Concrete normally contains air voids, accidentally or purposely entrained, distributed throughout the paste component. Normally the air voids, coarsest of all the pores, may constitute from less than 1 percent to more than 10 percent of the total volume of concrete. Many of the accidentally entrapped voids are usually larger in size when compared to the purposely entrained voids, and may range in size up to 2 mm (20). The entrained air voids range in size from about 0.01 to 0.5 mm. According to Neville (18), the entrained air produces discrete cavities in the cement paste so that no channels for the passage of water are formed and the permeability of the concrete is not increased. These voids never become filled with products of hydration of cement as gel can form only in water (18).

2.1.4 Aggregate Pores

The porosity of common aggregate materials varies from 0 to 20 percent of the solid volume of aggregates. However, for common aggregates the porosity varies from about 1 to 5 percent (20). The size of the pores varies greatly according to the type of aggregate. In dense aggregates they are in the size range of paste capillary spaces of intermediate size. Frequently the aggregate pores are at least the size of the largest capillary pores (20).

2.1.5 Pores and Voids in Concrete

On an approximate basis a normal concrete may contain about 75 percent of aggregate by volume, whose internal pore volume may vary from 1 to 5 percent of the solid volume of aggregates. The remaining 25 percent of the volume is made up of cement paste consisting of unhydrated cement grains, cement gel, capillary pores, gel pores, and air voids. It is also to be noted that the amount of capillary and gel pores available at any given time is largely dependent on the degree of hydration of the concrete. Normally it takes about a year for complete hydration of cement when the water-cement ratio is 0.70 (23, 24). A diagrammatic representation of the composition of concrete of water-cement ratio 0.70 is shown in Figure 2.

2.2 Pore Pressure in Soil Mechanics

Terzaghi (10) used the term pore pressure to define the hydrostatic excess pressure exerted by water which fills the voids of a saturated soil during the consolidation process. Soil is a multiphase system; a load applied to a soil mass is shared by the mineral skeleton and the pore fluid. The sketches in Figure 3 explain the gradual process of load transfer from pore fluid to the soil skeleton. In the analog the resistance of the mineral skeleton to the compression is represented by the spring. The resistance to the flow of water through the soil is represented by a valve in an otherwise impermeable piston.

Initially the entire applied load is assumed to be carried by the water when the valve is closed and the spring does not compress. When the valve is opened and as the water escapes, the spring shortens and begins to carry a portion of the applied load. In the final stage the entire load is carried by the spring and the pressure in the water reaches the external hydrostatic pressure.

Sharing the load between the mineral and pore phases also occurs in actual soil problems, although the pore fluid will not always carry all

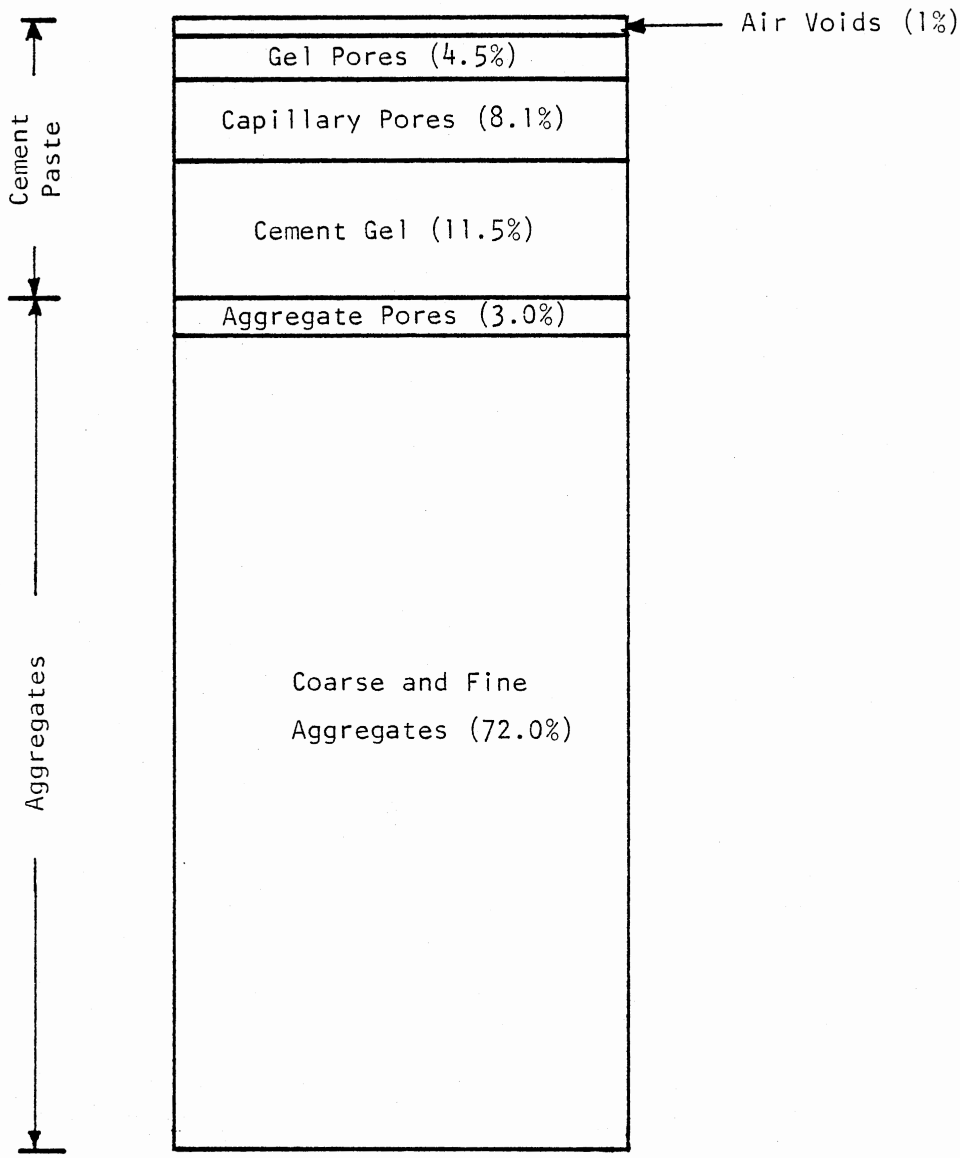
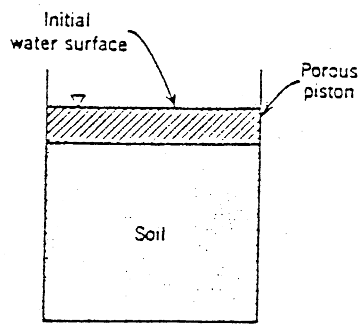
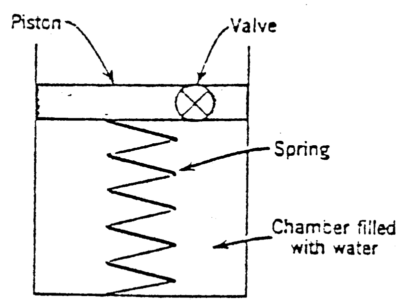


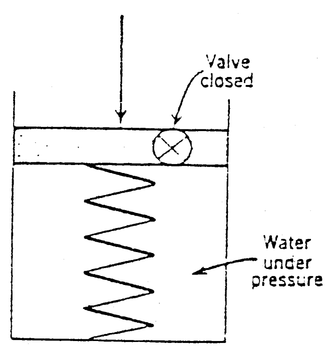
Figure 2. Approximate Volumetric Proportions of Concrete of Water-Cement Ratio 0.7 at 100% Hydration



a. Saturated Soil Mass

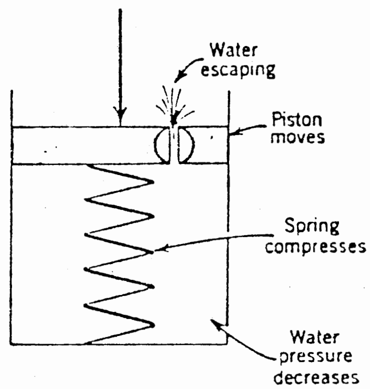


b. Hydromechanical Analog

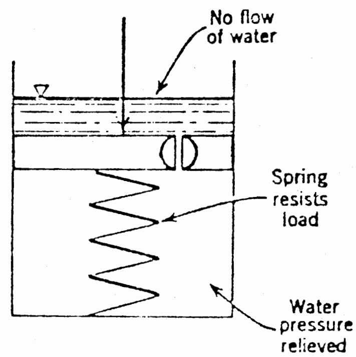


c. Load Applied With
Valve Closed

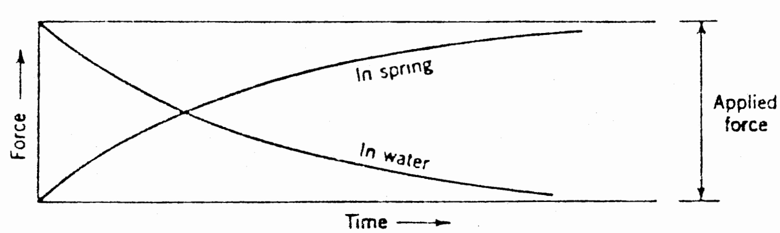
Figure 3. Piston and Spring Analog for Load Sharing (25)



d. Piston Moves as Water
Escapes



e. Equilibrium With No
Further Flow



f. Gradual Transfer
of Load

Figure 3. (Continued)

of the applied load initially. However, as indicated in Figure 3e, the process of load transfer is gradual. The magnitude of initial excess pore pressure produced at a given point in a soil skeleton depends on various pore pressure parameters.

2.2.1 Pore Pressure Parameters

Skempton (26) conveniently expressed the change in pore pressure Δu_w occurring under changes in the principal stresses $\Delta\sigma_1$ and $\Delta\sigma_3$ as

$$\Delta u_w = B(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)) \tag{2.1}$$

where A and B are pore pressure parameters. A change in pore pressure can be considered as taking place in two stages, first from a change in all-around stress of $\Delta\sigma_3$ and second from a change in deviator stress of $(\Delta\sigma_1 - \Delta\sigma_3)$. Hence Δu_w can be expressed as

$$\Delta u_w = \Delta u_a + \Delta u_d \tag{2.2}$$

where Δu_a and Δu_d are, respectively, the changes in pore pressure resulting from changes in all-around stress and deviator stress.

Equating the total volume change of the soil skeleton to the change in volume of the void space, Skempton (26) derived expressions for the pore pressure parameters as

$$B = \frac{\Delta u_a}{\Delta\sigma_3} = \frac{1}{1 + \frac{nC_v}{C_c}} \tag{2.3}$$

and

$$\Delta u_d = BA\ (\Delta\sigma_1 - \Delta\sigma_3) \tag{2.4}$$

where n is the porosity of the soil, C_v is the compressibility of the fluid (air and water), and C_c is the compressibility of the soil structure. Both parameters A and B are obtained experimentally from undrained triaxial compression tests on soils.

It may be observed that the magnitude of the parameter B depends on the degree of saturation and varies from a value of one for fully saturated soils to zero for completely dry soils. The parameter A is very much influenced by the strain, initial stress on the system, stress history, and type of stress change (25). A total stress increment on a soil element causes a pore pressure increment within the soil and the magnitude of the pore pressure increment developed depends on the nature of soil, type of stress, magnitude of strain, and time.

2.3 Effective Stress Concept

Figure 4 represents qualitatively a typical cross section through two soil grains in contact. For purposes of the following discussion, the two grains are shown to be sand and molecular or interparticle forces are not taken into account.

The total area of soil subjected to a load P is A_G; the two grains are in actual physical contact over a small area A_s, and the areas of contact between water and soil and air and soil are A_w and A_g, respectively. The stresses in solid, liquid, and gaseous phases are σ_s, σ_w, and σ_g, respectively. For equilibrium we have

$$P = \sigma_s A_s + \sigma_w A_w + \sigma_g A_g \qquad (2.5)$$

Dividing both sides of the equation by the gross area A_G, we get

$$P/A_G = \sigma = a\sigma_s + b\sigma_w + (1 - a - b)\sigma_g \qquad (2.6)$$

where σ is the total stress over the gross area, and a, b, $(1 - a - b)$ are

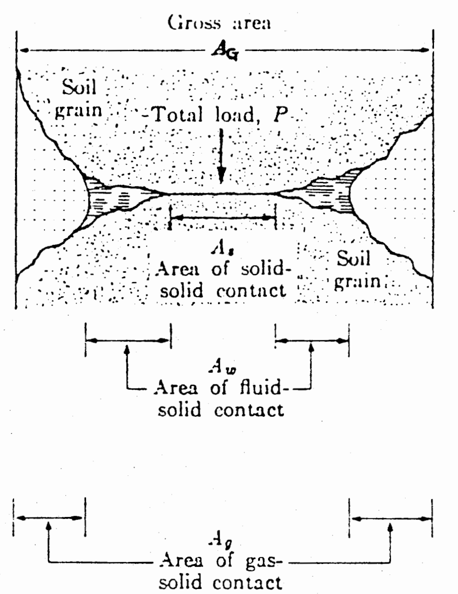


Figure 4. Intergranular or Effective Stress on Area (27)

the ratios of the areas of solid, liquid, and gaseous contact with the solid, to the gross area A_G. The parameter b is related to the degree of saturation of the soil. Equation (2.6) may be expressed in an alternate form as

$$\sigma = a\sigma_s + (1 - a)\sigma_w + (1 - a - b)(\sigma_g - \sigma_w) \qquad (2.7)$$

where the terms in parentheses represent equivalent pore pressures due to the pressures in the liquid and gaseous phases. If the soil is completely saturated, then (1 - a - b) equals zero and Equation (2.7) becomes

$$\sigma = a\sigma_s + (1 - a)\sigma_w \qquad (2.8)$$

In soils the ratio a of the area of actual solid contact to total area will be very small so that the term (1 - a) approaches unity. However, σ_s, the stress in the solids part of the interface, is very high and the product $a\sigma_s$ does not become equal to zero. The quantity $a\sigma_s$ is defined as the effective stress in the soil skeleton and is denoted by σ'. Thus Equation (2.8) can be written as

$$\sigma = \sigma' + \sigma_w \qquad (2.9)$$

or

$$\sigma' = \sigma - \sigma_w \qquad (2.10)$$

where

$$\sigma_w = u_w + \gamma_w h \qquad (2.11)$$

in which h is the steady-state pressure head in the pore water, u_w is a transient pore water pressure in excess of the steady-state pressure, and γ_w is the unit weight of water. In a laboratory test the sample is normally so small that $\gamma_w h$ may be neglected in comparison to the applied stress. Hence Equation (2.10) can be written as

$$\sigma' = \sigma - u_w \tag{2.12}$$

where σ' is the effective stress, σ is the total stress, and u_w is the pore water pressure.

Terzaghi (10) was the first to propose the above equation for effective stress based on his experiments on saturated soils. It is called the law of effective stress or the effective stress concept. This concept is widely used in soil mechanics and is regarded as satisfactory for all types of soil.

2.4 Triaxial Tests

The triaxial test has been very highly developed as a method of studying the mechanical properties of soil. The advantages of the triaxial tests are the control of drainage conditions and the ability to measure pore pressure. In most standard tests the application of all-around pressure and deviator stress form two separate stages of the tests. Tests are therefore classified according to conditions of drainage obtained during each stage (15). In undrained tests no drainage and hence no dissipation of pore pressure is permitted during the application of the all-around stress as well as during the application of the deviator stress. In consolidated-undrained tests drainage is permitted after the application of the all-around stress so that the sample is fully consolidated under the stress. No drainage, however, is permitted during the application of the deviator stress. The application of the deviator stress induces pore pressures which are measured and the effective stress on the sample is obtained using the effective stress law. However, the rate of deformation must be slow enough to allow the induced pore water pressures to distribute themselves evenly throughout the sample. In drained tests drainage is permitted throughout the test so that full consolidation occurs under the all-around stress and no excess pore pressure is set up during the application of the deviator stress. Pore pressures are not measured during this test since all stresses are effective stresses. The rate of deformation has to be very slow and varies with the type of soil, and is usually of the order of 0.01 percent per minute. Bishop and Henkel (15) have described in detail the instrumentation required and the techniques used in triaxial tests.

2.5 Pore Pressure in Rocks

While both soils and rocks can be classified as porous media, there is considerable difference in the nature of the medium. Soils are usually regarded as consisting of discrete particles touching at isolated points, while rocks and concrete are regarded as a solid skeleton traversed by a fine network of capillaries (28). The effect of fluid pressure in the pores of a rock and its movement through the pores is of practical importance. The introduction of the concepts of pore pressure and effective stress by Terzaghi (10) to soil mechanics stirred considerable interest in geologists and geophysicists who investigated the relevance and applicability of the above concepts to rocks of different porosities. Major studies on rocks have been made by Handin et al. (29), Aldrich (30), and Byerlee (31), and their results are in general agreement with the statement that, provided the rocks have connected systems of pores, fracture is controlled by the effective stress.

The stress conditions for the failure of saturated rock and concrete are discussed by Terzaghi (9). He states that the influence of pore water pressure on confined compressive strength of rocks can be determined only by means of triaxial compression tests with controlled pore pressure conditions. Robinson (32) conducted drained compression tests with controlled pore pressures on saturated sedimentary rocks. Drained tests are tests in which the applied pore pressure is kept at a constant preselected level. He found that the mode of failure of the rocks was controlled by the difference between the confining and pore pressure. He also stated that brittle failure always occurred when the confining and pore pressures were equal, and that the mode of failure changed gradually from that of a brittle material to a malleable (ductile) material as the differential between the confining and pore pressures increased. Handin et al. (29), from their drained triaxial tests on Berea sandstone, limestone, and shale at different confining pressures and pore pressures, concluded that the mechanical properties, ultimate strength, and ductility are functions of the effective stress, the effective stress being obtained by subtracting the pore pressure from the applied external stress. They stipulated, however, that the rock be a sandlike aggregate with connected pore space and sufficiently permeable so as to permit the free flow of interstitial fluid in or out of the rock during deformation. They said that this ensures that the pore pressure is transmitted fully throughout the solid phase and it remains constant and uniform throughout the sample. Byerlee (31), from his drained triaxial tests on Weber sandstone, found that both the fracture strength and frictional strength obeyed the effective stress law. That is, the strengths were determined by the effective stresses and not the total stresses acting on the sample.

Although the pore pressure effects on rocks have mainly been studied by means of drained triaxial tests, several authors have conducted undrained triaxial tests (30, 33). In undrained tests the specimen drainage is closed off at the start of the test and the changes in pore pressure within the specimen induced by the loading are measured. Heck (34) describes in detail the apparatus used and the procedure for conducting undrained triaxial compression tests on saturated rocks with measurement of changes in pore pressure due to loading. Aldrich (30) verified the effective stress law from drained and undrained triaxial tests conducted on Berea sandstone. He also found that the strength at failure of an undrained test specimen is controlled by the effective confining stress at failure, and the magnitude of pore pressure rise in the specimen is governed by the initial effective confining pressure.

Bieniawski (35) proposed a hypothesis on the mechanism of rock fracture outlining all failure processes taking place in rock from initial load application to complete failure. He also provided experimental verification of the postulated mechanism of brittle fracture of rock. The brittle fracture is defined as fracture that exhibits little or no permanent deformation, implying that material behaves elastically (but not necessarily linearly) up to fracture. Lane (33) introduced two additional features--residual strength and induced pore pressure--to the fracture concept of Bieniawski, based on the behavior of rocks exhibited during triaxial tests. He diagrammatically represented the fracture concept as shown in Figure 5. It may be seen that the various stages of brittle fracture of rock are: initial closing of cracks, linear elastic

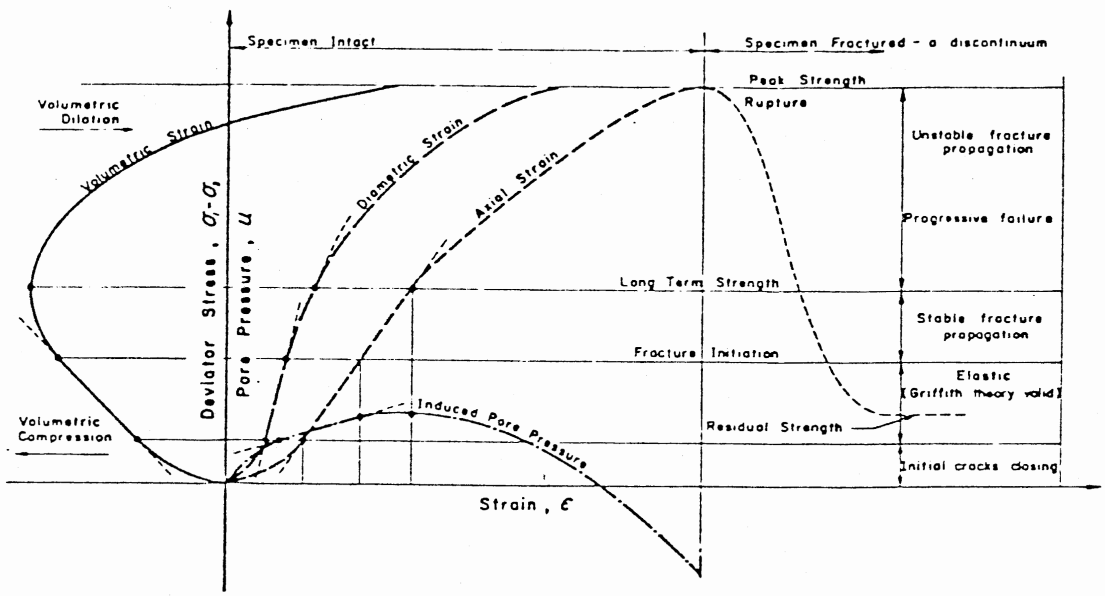


Figure 5. Fracture Concept of Intact Rock (34)

deformation, stable fracture propagation, unstable fracture propagation, strength failure, and rupture. Of special interest is the point where the axial strain curve departs from linearity which also brings about a change in sign of the volumetric strain from compression to dilation. The onset of dilation marks the beginning of reduced pore pressure and long-term strength. Stressing above such long-term strength results in progressive failure.

Lane also conducted undrained triaxial compression tests on Berea sandstone and diagrammatically presented the results as shown in Figure 6. Comparing Figures 5 and 6, the remarkable agreement between Bieniawski's theoretical concept as modified by Lane and Lane's experimental data may be seen. It may also be noted that the induced pore pressure reaches its maximum when the specimen has compressed to minimum volume and the induced pore pressure becomes negative (less than the initial back pressure) after the specimen has expanded more than its original volume.

The work done on sedimentary rocks was gradually extended to crystalline rocks of low porosity. The presence of pore pressure in very dense rocks is thought to be negligible though there is no general agreement as to where the division lies between rocks that are affected by pore pressure and rocks that are not (36). Brace and Martin (14) conducted drained triaxial tests on Westerly granite rock of very low porosity at constant and zero pore pressures and at different strain rates. Their results indicated that the strength of the rock with pore pressure was greater than that of rocks with zero pore pressure and the increase in strength became larger at more rapid strain rates. The increase in strength is reported to be as much as 50 percent over the value at zero

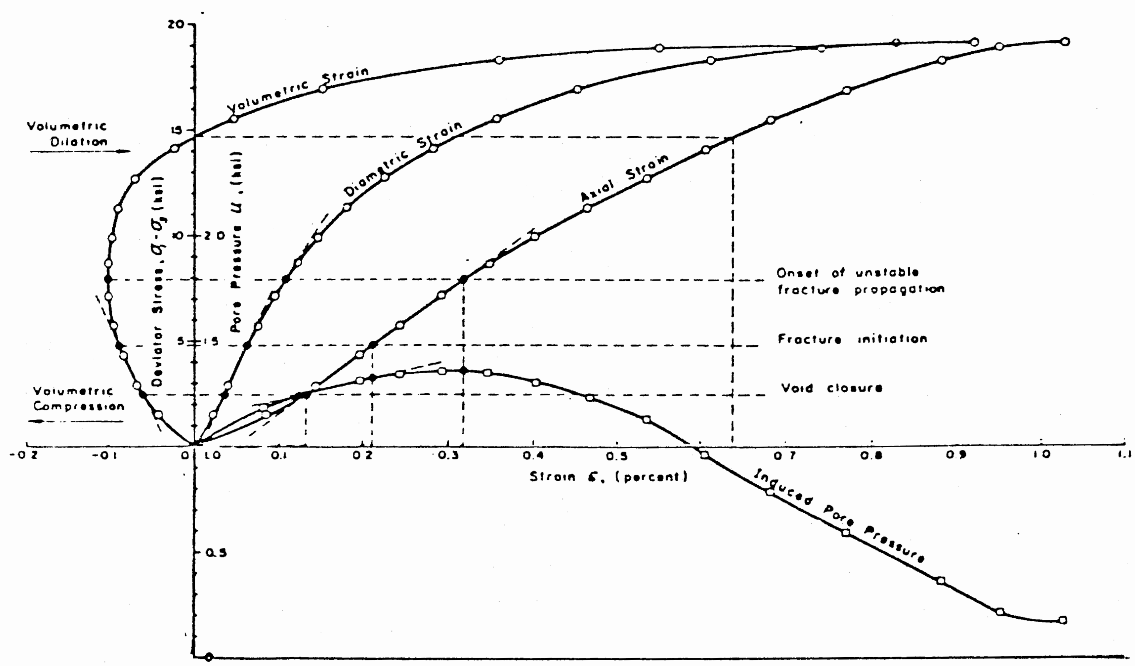


Figure 6. Typical Plot of Data From Undrained Triaxial Compression Test on Berea Sandstone (34)

pore pressure. The apparent strengthening effect of rapid loading is attributed to the phenomenon of dilatancy hardening. They explain that the rocks become dilatant prior to fracture, representing an increase in porosity and thereby reducing the pore pressure within the sample, even though the pore pressure was maintained constant on an external gage during loading. At fast loading rates they maintained that changes in pore pressure lagged behind changes in the external part of the pore fluid system. Hence the drop in pore pressure within the sample had the effect of increasing the effective confining stress and caused the increase in strength. The more rapid the loading rate, the greater the drop in pore pressure and consequent increase in strength. Thus the strain rate became a critical parameter influencing the strength of rocks with pore pressures. Brace and Martin concluded from their experiments that at the critical strain rate the changes in pore pressure in the sample just keep pace with those in the external system, and only up to the critical strain rate the law of effective stress holds good. The value of the critical strain rate was reported to be 10^{-7} to 10^{-8} per second for most of the rocks. They also stated that the critical strain rate depends not only on the particular rock but also on the fluid in the pores and geometrical factors such as the distance from the center of the rock mass to a source of pore fluid.

Duba et al. (37) tested jacketed saturated samples of Westerly granite of 0.8 percent porosity with various amounts of excess water placed inside the jacket containing the initially saturated specimen and sealed. The amounts of excess water ranged from 0.2 to 19.2 percent of the total volume of rock sample. They found that the results obtained were consistent with the concept of effective stress. A strain rate of about 10^{-4}

per second was used in their experiments. They also observed that the strength of samples with 20 percent water--that is, 19.2 percent of excess water added to the already saturated sample--was independent of the confining pressure outside the jacket. The strengths of the rest of the samples, with varying amounts of excess water, were found to increase with decreasing amounts of water available within the jacket. Hence they concluded that if the granite is saturated and with no extra water available to maintain pore pressure, it is as strong as a dry rock; but with sufficient water available to maintain pore pressure, it can be considerably weakened. Therefore, the availability and transfer of water in the immediate vicinity become important variables in evaluating the strengths of saturated rock.

Nur and Byerlee (38) theoretically developed an effective stress law for elastic deformation of rock with fluids. In deriving the law, the following assumptions were made:

1. The aggregate is mechanically isotropic and elastically linear, permitting superposition of strains.

2. The deformation of the porous material can be treated in the same way as the deformation of nonporous elastic solids so long as pore pressure is not considered.

3. Strains are linearly related to pore pressure.

Using the above, an expression for the effective stress, not directly dependent on pore configuration or porosity of rocks, was derived as

$$\sigma' = \sigma - (1 - (K/K_s))u_w \tag{2.13}$$

where K is the bulk modulus of the dry aggregate, and K_s is the intrinsic bulk modulus of the solid grains in the aggregate. They also stated that the above law is not automatically applicable to an inelastic process such as fracture. However, Skempton (8) had empirically proposed the same expression for effective stress in rocks and concrete much earlier. It may be seen that the above expression for effective stress differs from Terzaghi's effective stress law only in that the pore pressure is multiplied by a factor $(1 - (K/K_s))$. However, when the compressibility of the soil grain is negligible when compared to the compressibility of the entire soil skeleton, the term (K/K_s) equals zero and the law proposed by Nur and Byerlee reduces to the simple expression for effective stress proposed by Terzaghi.

2.6 Pore Pressure in Concrete

2.6.1 General

The earliest hydraulic structures to be affected by the pressure of interstitial water were dams. Stability failures of a number of dams early in this century generated interest in the phenomenon of uplift, a force exerted on the body and foundation of a dam by the percolating water.

2.6.2 Uplift

Uplift exists in dams when the interconnected pore spaces in the concrete become filled with water under pressure. According to earlier theories, the uplift pressure was due to and the consequence of a hypothetical crack through which the water penetrated into the dam (7). It is now widely accepted that water percolates through the body of the dam, inducing uplift pressures (4, 7, 39). The intensity of uplift pressure in a dam, dependent upon the permeability of the concrete and the length of time the reservoir remained filled, has been a matter of controversy (4). In stability analysis of dams it is usual practice to assume the uplift pressure to vary linearly from the water pressure at the upstream face to the water pressure at the downstream face (4). In view of the low permeability of well designed mass concrete, Carlson (3) suggested that equilibrium pore pressure would never likely be achieved in the life of a dam. However, there are several reports on uplift pressures which suggest the presence of appreciable water pressures throughout the dam (2, 11). The effective area on which the uplift pressure acts has been controversial and according to Terzaghi (9) it depends on the "boundary porosity" of the concrete.

2.6.3 Boundary Porosity of Concrete

The influence of a confining pressure on the strength of a specimen with empty voids can be investigated by means of triaxial compression tests on dry specimens protected by a water-tight membrane. Tests of this type were conducted by Richart et al. (40) for concrete and they obtained the approximate empirical relation

$$f_c = f'_c + 4.1\ \sigma_c \tag{2.14}$$

where f_c is the confined compressive strength, σ_c is the all-around confining pressure, and f'_c is the standard compressive strength of concrete. The reasons for the increase in strength of dry concrete specimen subjected to confining pressure, the reduction in strength of concrete with fluid in voids, and the definition of the term "boundary porosity" as furnished by Terzaghi (9) are given below.

The main reason for the increase in strength due to increase in the confining stress on the specimen with empty voids is due to the fact that the grains themselves are very much stronger than the bond between the grains; otherwise the failure would take place across the grains and, as a consequence, the strength would be practically independent of the confining pressure. The strength of the specimen depends on the stress which is carried by the bond between the grains along the potential surfaces of failure; the state of stress in the grains is irrelevant. If the voids of the specimen are empty, the normal stress due to a confining pressure, σ_c, is entirely carried by the solid material which constitutes the bond. The presence of fluid under pressure σ_c in the voids does not alter the stress, but it reduces that part of σ_c which is carried by bond and hence reduces the strength. The magnitude of the decrease depends on the degree of continuity of the intergranular bond. The ratio of that part of the area of potential surface of failure which is in contact with the interstitial liquid and the total area of this surface is referred to as "boundary porosity." Terzaghi (9) also mentioned that the data required for estimating the boundary porosity can be obtained by means of compression tests on saturated specimens with unprotected surface immersed in a liquid under pressure σ_c.

Terzaghi and Rendulic (16) conducted triaxial compression tests on concrete specimens with and without an impermeable jacket covering the specimens. Their series of experiments on unjacketed specimens showed that fluid pressures up to several hundred atmospheres had no measurable effect on the compressive strength of material. On the other hand, the above confining pressures acting on jacketed specimens increased the compressive strength considerably. To account for the above results, Terzaghi (9) proposed the value of boundary porosity of concrete to be very close to unity. He concluded that the area of actual contact between the individual grains could not have exceeded a small fraction of the total intergranular boundaries, and the volume porosity of the material being very low, the voids must consist of very narrow but continuous slits. In view of the high boundary porosity obtained for concrete, he held that the hydrostatic uplift in concrete is as active as in a cohesionless sand. He also suggested that the idea of considering uplift as acting only over a part of the area of concrete appeared to be erroneous. Laubscher (41) concurred with Terzaghi and argued that the boundary porosity of rocks should be equal to one.

McHenry (11) conducted triaxial compression tests, both jacketed and unjacketed, on about 330 samples, roughly equally divided in numbers between the sizes 2 in. by 4 in. and 6 in. by 12 in. The standard compressive strength of concrete used varied from about 4.5 to 6.0 ksi. Kerosene was used as the lateral pressure medium for the rubber jacketed speicmens, and nitrogen gas was used as the lateral pressure medium for the unjacketed specimens. He states that the use of a liquid for the lateral pressure medium would have required enormous time for the development of full pore pressure throughout the cylinder. He found that for jacketed specimens the axial strength increased by a factor equal to 4.7 to 6.9 times the lateral pressure. For the unjacketed specimens the increase in axial strength was roughly equal to the lateral pressure. He obtained values of boundary porosity ranging from 0.78 to 1.18, with the average being very close to one. Since the boundary porosity was approximately one, he concluded that the uplift pressure is effective over approximately 100 percent of the area of potential surface of failure.

Leliavsky (6) conducted tests on 95 concrete and mortar specimens covering a wide range of cement contents, water-cement ratios, and ages to determine the boundary porosity. The cylindrical specimens were 6 in. in diameter and had a 0.8 in. diameter central bore along their longitudinal axis. These unjacketed cylinders were subjected to various combinations of circumferential hydraulic pressure and axial load until tensile failure of concrete occurred. Based on his experiments, he reported an average value of 0.91 for the boundary porosity of concrete.

From a knowledge of the size and arrangement of gel particles, Powers (13) tried to estimate a possible area factor for cement gel when it is subjected to hydraulic pressure. He suggested that the upper limit for the area factor of cement gel may be very close to unity, though the actual area factor can be obtained only by experiment. Recently Butler (12) subjected unjacketed concrete cylinders to all-around hydrostatic pressure and axial tensile load until tensile failure of concrete occurred. He calculated the effective porosity of the concrete at various loading stages from the start to the failure load. He found that the effective porosity or boundary porosity was initially less than unity, but increased with increasing tensile stress, to unity at failure. He also observed that his results seemed to indicate a link between boundary porosity and water-cement ratio. He found the average effective porosities of concrete of water-cement ratios 0.47, 0.59, and 0.71 to be 0.62, 0.67, and 0.81, respectively. Harza (5) showed by deductive reasoning that boundary porosity has to be always unity for any concrete assumed to be porous.

2.6.4 Significance of Pore Pressure and the Law of Effective Stress

Terzaghi (9) held that the law of effective stress proposed for fully saturated soils was valid for concrete. For saturated concrete he modified Equation (2.14) proposed by Richart et al. for the confined compressive strength of concrete as

$$f_c = f'_c + 4.1\ (\sigma_c - u_w) \tag{2.15}$$

where $(\sigma_c - u_w)$ may be defined as the effective confining stress. From a limited number of drained triaxial compression tests on concrete, McHenry (11) found the effective stress law to be valid. Skempton (8) sought an explanation for the paradoxical behavior of concrete in having a very high boundary porosity and obeying the effective stress law in spite of being a material of very low porosity and permeability. He considered a wide range of triaxial tests on minerals, rocks, and concrete, and suggested that a more precise form of the effective stress law for a fully saturated porous material may be expressed as

$$\sigma' = \sigma - [1 - (a \tan \psi / \tan \phi)]u_w \tag{2.16}$$

where a is the ratio A_s/A_G as defined in Equation (2.6), ψ is the angle of internal friction of the solid substance comprising the material, and ϕ is the angle of internal friction of the entire porous material. Skempton calculated that the ratio $\tan\psi/\tan\phi$ had a value of 0.18 for concrete. It may be seen that when the boundary porosity of the concrete is one, the term "a" goes to zero and Equation (2.16) reduces to the simple effective stress Equation (2.12) proposed by Terzaghi. However, both

Skempton (8) and Butler (12) have held that the term "a" may not be negligibly small for rocks and concrete at stresses other than failure.

2.7 Effect of Degree of Saturation

So far the effects of pore pressure on a fully saturated porous medium have been reviewed. When the pore space of a soil contains both air and water, the soil is said to be partially saturated and the degree of saturation is defined as the ratio of volume of water to the volume of pore space. Though the concepts discussed so far for the fully saturated medium are fundamentally applicable to partially saturated soils, the formulae developed so far should be suitably modified for partially saturated soils. Bishop (42) suggested the effective stress equation for partially saturated soils as

$$\sigma' = \sigma - (u_a - b(u_a - u_w)) \qquad (2.17)$$

where b as defined in Equation (2.6) is the ratio of area of liquid in contact with the soil to the total gross area, and u_a is the pore-air pressure. The above equation was also experimentally verified by Bishop. It is evident that the parameter b is dependent on the degree of saturation and is equal to one when the degree of saturation is 100 percent and zero when the degree of saturation is zero. A special case arises when u_a is equal to the atmospheric pressure. Since all pressures are normally expressed in relation to atmospheric pressure as a base, u_a is then zero and

$$\sigma' = \sigma - bu_w \qquad (2.18)$$

Special techniques are needed to measure these parameters in partially saturated soils and the value of b must be experimentally obtained for a given degree of saturation (15). In view of the complexities involved, the influence of pore pressure on partially saturated rocks and concrete has not been studied and little is known on its effect on the mechanical properties of partially saturated rocks and concrete.

2.8 Fatigue of Plain Concrete

2.8.1 General

Fatigue is a process of progressive, permanent internal structural change in a material subjected to repetitive stress. Despite the early interest in metal fatigue, fatigue studies on concrete did not start until the early 1900's.

2.8.2 Previous Investigations on Fatigue of Concrete

Van Ornum (43) in 1903 was one of the earliest investigators to conduct fatigue tests on cubes of neat cement. He conducted similar tests on concrete prisms in 1907 (44). In the years that followed several investigations were carried out along the same lines. Nordby (45) and Murdock (46) have critically reviewed the work on fatigue of concrete. Though the state of knowledge on the fatigue aspects of concrete is rather limited, the following conclusions were arrived at by Murdock (39) based on the results reported in the literature on axial fatigue investigations.

1. Failure in fatigue occurs in plain concrete subjected to repeated axial compressive loads.

2. No fatigue limit exists for the material up to ten million cycles of load.

3. The fatigue response when expressed in terms of static ultimate strength is statistically independent of nominal strength, type of aggregate, air entrainment, or frequency of repetition of load.

4. The fatigue strength appeared to be dependent on the range of applied stress and increases as the range of stress is reduced.

5. The fatigue strength at ten million repetitions of load may be taken at 55 percent of the static ultimate strength.

6. The failure of concrete under repetitive loading appears to be progressive.

Hilsdorf and Kesler (47) were the first to investigate the effects of rest periods on the fatigue response. The inclusion of rest periods was found to be beneficial but duration of the rest periods was significant only for periods between one and five minutes. They did not observe any increase in the fatigue strength above a rest period of five minutes. Bennet and Muir (48) conducted fatigue tests on concretes of strengths 6000 and 8500 psi and found the general behavior comparable with that observed in earlier tests. They reported the fatigue strength after one million repetitions to vary between 66 and 71 percent of the static strength. Weigler and Klausen (49) monitored the damage to the concrete specimens due to cyclic compressive loading using acoustic emission equipment. They concluded that the damage process can be divided into three stages. Stage I signifies the appearance of microcracks. In Stage II no significant increase in damage takes place. Stage III indicates an unstable crack growth leading to failure. They observed that the transition from stage II to stage III occurred at approximately 70 percent of the number of cycles to failure.

A report by ACI Committee 215 (50) discusses the considerations for design of concrete structures subjected to failure loading. The fatigue strength at ten million cycles of load is reported to be 55 percent of the static ultimate strength. The above fatigue strength is for a probability of failure of 50 percent and is valid regardless of the type of loading, compression, tension, or flexure (50). The fatigue strength of mortar and concrete when expressed as a percentage of their corresponding ultimate strength are about the same (51).

2.8.3 Fatigue Tests on Saturated Concrete

The effect of saturation on fatigue characteristics is unknown. Hatt (52) conducted flexural fatigue tests on saturated mortar beams. The mortar beams were saturated by immersion prior to testing. The first series of beams were immersed for 200 hours prior to testing and the second series were kept continuously immersed for 4 months. The fatigue strengths for series one and series two were found to be 37 and 45 percent of the static strength of dry mortar beams. The fatigue strength of companion mortar beams tested in an air-dry condition was reported to be 55 percent of static strength of air-dry mortar beams. Hatt also reported that the static strengths of the saturated mortar specimens used in test series one and two were, respectively, 89 and 83 percent of the strength of dry mortar. However, it should be pointed out that the fatigue strength of 45 percent reported for series two is approximately 55 percent of the static strength of similarly saturated beams. Hence there appears to be no noticeable difference in fatigue strengths of dry and saturated specimens, provided the fatigue strength of a saturated specimen is expressed as a percentage of the static strength of a similarly saturated specimen.

Kaplan (19) studied the effect of moisture and the rate of loading on the strength of concrete. He found that the strength of moist specimens increased with increasing loading rate, whereas the strength of the dry specimens was not affected by the rate of loading. He explained that at high rates of loading the pore pressures developed in the concrete proved to be beneficial in reducing the effective stress applied to the concrete. He also found that the increase in strength was more pronounced in the case of older, well hydrated specimens when compared to very young specimens. He states that matured specimens have smaller pores and consequently develop high pore-water pressures, while younger specimens which have larger pores due to incomplete hydration develop comparatively lower pore pressure.

Guidelines for the design of fixed offshore concrete structures are discussed in a report by ACI Committee 357 (53). The report states that the resistance of concrete to fatigue is considered adequate if the stresses in compression, flexural tension, and membrane tension are limited to 0.5 f'_c, 200 psi, and 0 psi, respectively. Waagaard (54) suggested that in submerged concrete structures, the water in the crack could be subjected to temporary high pressures during flexural fatigue loading. Gerwick (55) stated that the water in the crack under instantaneous hydrostatic pressure peak could cause hydraulic fracturing of the concrete.

2.9 Summary

Pore structure of concrete in terms of size, continuity, and volume is analogous to that of some rocks. Several studies have shown that significant pore pressure could develop in saturated rocks and concrete when subjected to confining pressures or triaxial compression. Convincing evidence is also available in the literature to assume the value of boundary porosity or effective porosity of concrete to be very close to unity. As a consequence the pore pressure may be assumed to be effective over 100 percent of the area of the potential surface of failure. Several experimental investigations have confirmed the validity of the effective stress law, originally proposed for saturated soils, for saturated concrete.

Saturated rocks, dense granite as well as fairly porous sedimentary rocks, have been found to obey the effective stress law. Brace and Martin (14) observed an increase in strength of saturated dense granite when subjected to rapid load rates. The increase in strength is said to be due to dilatancy hardening. Negative pore pressures developed in the sample due to dilation or increase in pore volume cause an increase in effective confining stress and consequent increase in strength. Duba et al. (37) found that the apparent increase in strength of the rock due to dilatancy hardening was either lost or reduced by the presence of a close recharge source to the saturated specimen.

Bieniawski's (33) concept on fracture of intact rock and Lane's (34) experimental verification of the same provide useful information on the reduction in pore pressures at the onset of volumetric dilation of rocks. Though concrete may be expected to behave in a similar fashion, no experimental evidence is available on the aspect of dilatancy hardening or on the general effects of pore pressure on the mechanical properties of concrete. Waagaard (54) and Gerwick (55) mention the possibility of high pore pressures developing in submerged structures that could cause hydraulic fracturing of the concrete. Kaplan's (19) experiments on the influence of load rates on moist concrete specimens give an indication of the presence of a problem. A suddenly applied load to a saturated concrete may either be carried by the pore fluid or jointly by the pore fluid and the porous skeleton. The pore pressures developed within the specimen, not necessarily uniform throughout the cross section, may depend on several variables connected with pore and fluid characteristics.

The effects of pore pressure on the mechanical properties of concrete may be expected to be more pronounced in the case of saturated specimens subjected to repeated axial loadings. The high pore pressures developed repeatedly during loading could initiate microfracturing in the concrete and start the "damage" process in fairly low number of cycles when compared to dry concrete. Only one investigator has conducted limited fatigue tests on saturated cement mortar (52). These tests were carried out in the early 1920's and were not intended to study the effects of pore pressure, a concept just then developed in the area of soil mechanics. A report of the ACI Committee 357 (53) suggests limiting stress values in compression, flexural tension, and membrane tension for concrete used in submerged concrete structures subjected to fatigue loadings. The suggested limiting stress values do not appear to have been based on experimental data for saturated or submerged concrete subjected to fatigue loading.

CHAPTER III

EXPERIMENTAL PROGRAM

3.1 Introduction

The program described below was intended to investigate the effects of pore pressure on concrete as manifested in the fatigue characteristics of saturated concrete, tested under moist conditions. Specimens used in this study consisted of saturated cement mortar cylinders of sizes 3 by 6 inches and 4 by 8 inches.

3.2 Materials

3.2.1 Cement

Type I portland cement purchased locally was used in mixes for all the specimens.

3.2.2 Aggregate

Fine aggregate used in the mix was a mixture of natural coarse sand and masonry sand. The ratio by weight of coarse sand to masonry sand was 3:1. The coarse sand which met the gradation requirements for fine aggregate had a fineness modulus of 2.86 and a specific gravity of 2.63. The masonry sand which was added to improve workability and to reduce bleeding had a fineness modulus of 1.83 and a specific gravity of 2.64.

3.2.3 Cement Mortar

The mix proportions by weight of cement and the blended sand were 1.0:5.0. The water-cement ratio was 0.70. The nominal design strength of the mix was 3000 psi.

3.3 Experimental Procedure

3.3.1 Casting and Curing

The mortar was mixed in approximately 500 lb batches for about ten minutes using a pan mixer with a capacity of 3.75 cubic feet. All specimens of a given size were cast from a single batch of mix.

The specimens were cast vertically in disposable cardboard molds. The molds were filled in three lifts and consolidated using a table vibrator. The specimens were covered with a plastic sheet for 24 hours at which time they were demolded and placed in a moist room.

The period of curing in the moist room ranged from 125 to 144 days for the 3 by 6 in. specimens and from 148 to 236 days for the 4 by 8 in. specimens. For a water to cement ratio of 0.70 and a minimum curing period of 125 days the degree of hydration of the cement may have been as high as 95 percent. Hence the pore structure of the concrete had reached a high degree of stability at the end of moist curing.

3.3.2 Saturation Procedures

On removal from the moist room, the specimens were oven dried to constant weight at a temperature of 95 to 100°C. The period of oven drying ranged from about 100 to 110 hours for the 3 by 6 in. specimens and from about 120 to 150 hours for the 4 by 8 in. specimens. The oven dried specimens were then capped at both ends using a sulfur based compound.

The saturation apparatus consisted of a suitable pressure container and a water supply arranged as schematically shown in Figure 7. The water used for saturating the specimens was first deaired for several hours. The capped oven dried specimens were placed in the pressure container and evacuated by drawing a vacuum of about 29 in. of mercury for eight hours. Deaired water was then permitted to enter the container over a period of about 30 minutes; after this time the specimens in the pressure container were fully immersed. The weight of each specimen under suface dry condition was determined after about 48 hours of immersion in the pressure container. The specimens were then placed in water and kept immersed until time of test. Just prior to testing each specimen was again weighted to provide data on the additional saturation of the specimens resulting from immersion after the initial 48 hours of immersion. The dimensions and the various weights of the specimens are provided in the Appendix.

3.3.3 Fatigue Testing

Fatigue tests were conducted on seventy-one 3 by 6 in. cylinders and forty-eight 4 by 8 in. cylinders. Prior to the initiation of fatigue tests three 3 by 6 in. specimens and five 4 by 8 in. specimens were loaded to failure using a stress rate of 30 psi per second. The average static strength, f'_c, from these tests were used as a reference in subsequent fatigue tests.

The minimum stress levels in all fatigue tests were 0.096 f'_c for the 3 by 6 in. specimens and 0.1 f'_c for the 4 by 8 in. specimens. The

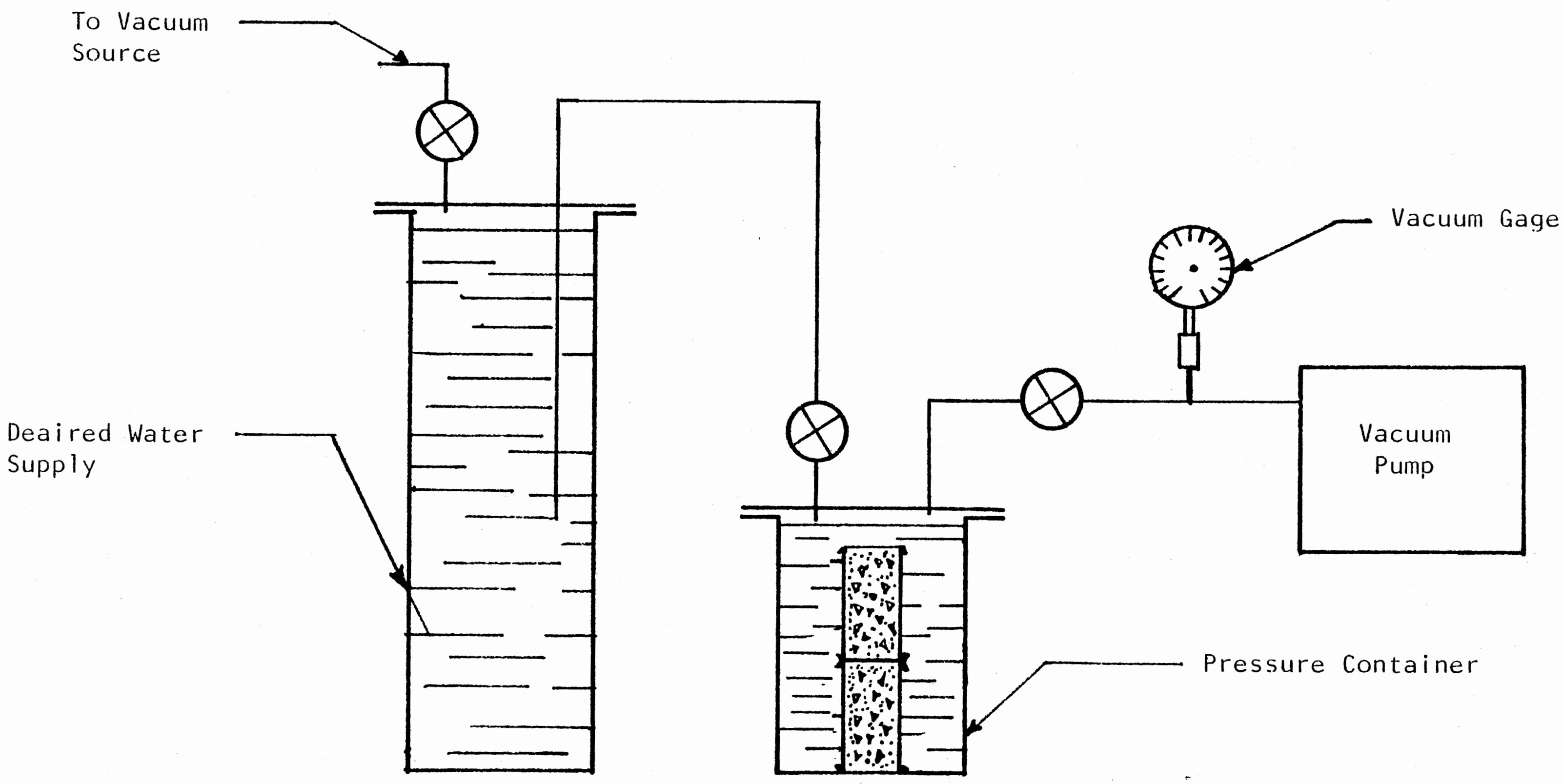


Figure 7. Saturation Apparatus

maximum stress level was 6, 7, or 8 times the minimum stress level. Fatigue loading was sinusoidal and applied at a frequency of one or ten Hertz. Some specimens were subjected to a loading spectrum which contained rest periods between each load application. The various loading spectra are illustrated in Figure 8. The smaller cylinders were tested under one of ten possible load spectra involving frequency, peak stress and rest period; the larger cylinders were tested at a single stress range of 0.1 to 0.7 f'_c using four of the five load spectra.

Specimens were kept moist during fatigue tests. Approximately half of the specimens were tested in water. The remaining specimens were wrapped with a paper towel which was kept moist by water drawn by capillary action from a small quantity of water provided at the bottom of the container.

The cyclic compression loads were applied by a 100,000 lb capacity, servo-controlled, hydraulic testing machine. The age of the specimens at test ranged from 309 to 442 days for 3 by 6 in. specimens and from 448 to 464 days for 4 by 8 in. specimens. Table I gives the details of the test program for the 28 test series.

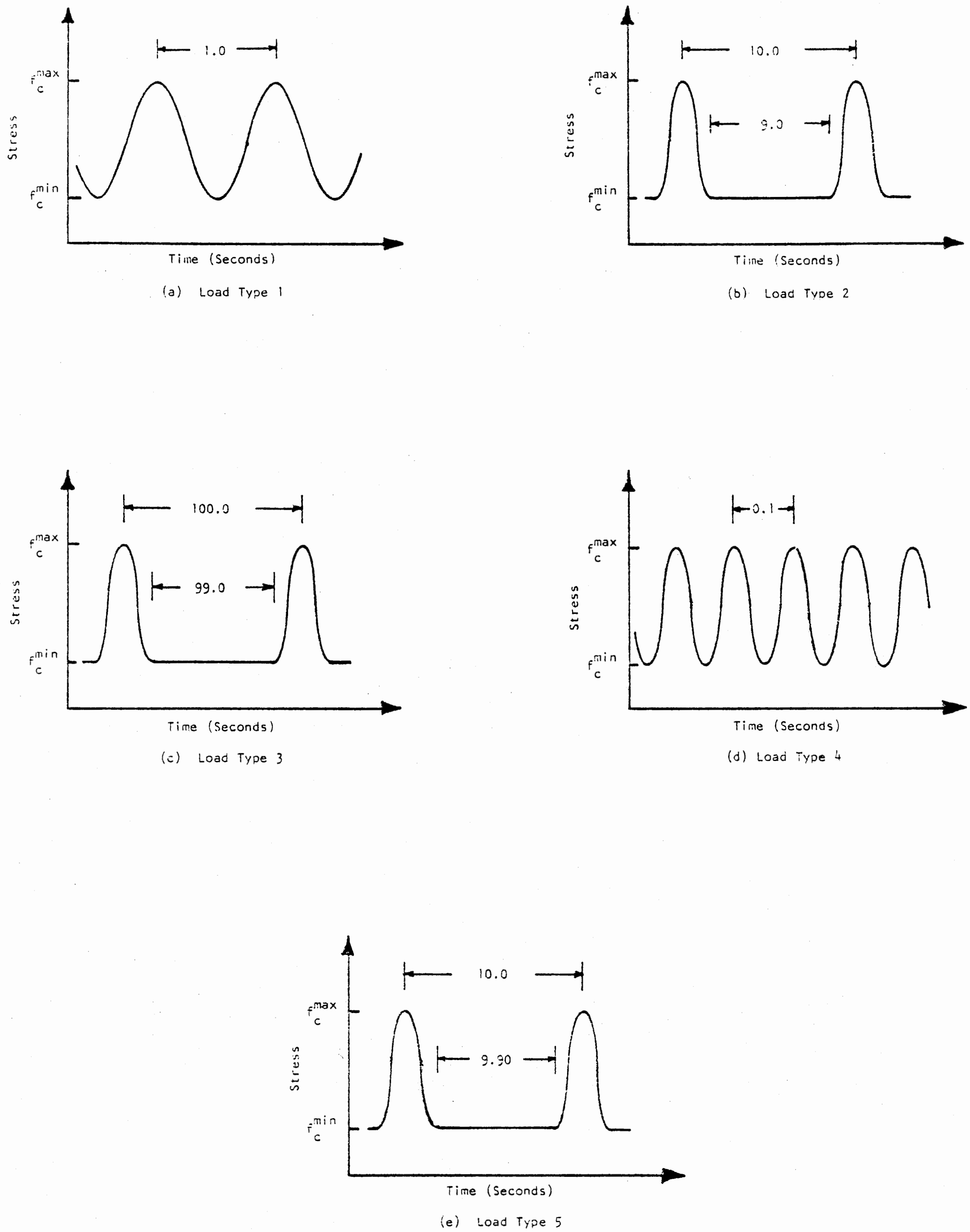


Figure 8. Load Configurations

TABLE I

TEST PROGRAM

Series	Load Type	$\frac{f_c^{max}}{f'_c}$	No. of Specimens	Frequency, Hz	Rest Period Sec.	Moisture Condition
		3 by 6 in. Specimens; Minimum Stress Level, 0.096 f'_c				
A	1	0.77	3	1	0	Submerged
AA	1	0.77	3	1	0	Moist
B	1	0.67	4	1	0	Submerged
BB	1	0.67	4	1	0	Moist
C	1	0.57	3	1	0	Submerged
CC	1	0.57	3	1	0	Moist
D	2	0.77	3	1	9	Submerged
DD	2	0.77	3	1	9	Moist
E	2	0.67	4	1	9	Submerged
EE	2	0.67	4	1	9	Moist
F	2	0.57	3	1	9	Submerged
FF	2	0.57	3	1	9	Moist
G	3	0.77	3	1	99	Submerged
GG	3	0.77	3	1	99	Moist
H	3	0.67	4	1	99	Submerged
HH	3	0.67	4	1	99	Moist
I	4	0.67	4	10	0	Submerged
II	4	0.67	5	10	0	Moist
J	5	0.67	4	10	9.9	Submerged
JJ	5	0.67	4	10	9.9	Moist
R	Static Strength Tests on 3 Specimens					
		4 by 8 in. Specimens; Minimum Stress Level, 0.1 f'_c				
K	1	0.70	5	1	0	Submerged
KK	1	0.70	6	1	0	Moist
L	2	0.70	6	1	9	Submerged
LL	2	0.70	7	1	9	Moist
M	4	0.70	7	10	0	Submerged
MM	4	0.70	5	10	0	Moist
N	5	0.70	6	10	9.9	Submerged
NN	5	0.70	6	10	9.9	Moist
S	Static Strength Tests on 5 Specimens					

CHAPTER IV

RESULTS

4.1 Saturation Tests

The data pertaining to saturation charactertistics of all the specimens are given in Table II. The volumetric water content as given in the table is the ratio of volume of ingressed water, obtained from the difference in weight between the saturated and oven dried weights of the specimen, to the total volume of the specimen, expressed as a percentage. The volumetric water content of each specimen at vacuum saturation and at test are given to provide data on additional saturation of the specimen as a result of immersion.

The average values of volumetric water content of 74 specimens of size 3 by 6 in. and 53 specimens of size 4 by 8 in. at vacuum saturation and at test are given in Table III. From the results it may be seen that the average values of volumetric water content at test for both sizes of specimens are roughly the same. However, the scatter of the data for the larger specimens is greater than that for the smaller specimens.

The immersion of the specimens, following the vacuum saturation for periods ranging from 138 to 300 days for 3 by 6 in. specimens and from 200 to 302 days for the 4 by 8 in. specimens, helped to considerably increase the volumetric water content of some of the specimens resulting in a lower coefficient of variation for the values of volumetric water content of the specimens. From the available gravimetric data, the

TABLE II

SATURATION DATA

Specimen	Age at Vacuum Saturation Days	Immersion Period Days	Volumetric Water Content At Vacuum Saturation %	Volumetric Water Content At Test %
		3 by 6 in. Specimens		
A1	132	177	23.7	27.4
A2	136	175	25.0	25.6
A3	142	167	24.7	25.4
AA1	132	179	26.2	27.8
AA2	132	179	26.9	27.7
AA3	138	175	25.3	26.1
B1	132	180	25.6	27.6
B2	136	290	24.8	25.7
B3	136	178	24.1	26.2
B4	142	170	25.1	25.5
BB1	132	180	24.1	27.1
BB2	136	178	22.7	25.2
BB3	138	174	18.3	24.6
BB4[a]	246	179	25.4	25.5
C1	132	199	26.1	26.9
C2	138	193	25.3	26.0
C3	142	190	20.3	23.7
CC1	132	216	21.4	26.2
CC2	142	206	25.5	25.9
CC3	152	197	26.1	26.6
D1	138	209	19.5	25.3
D2	142	205	25.6	26.0
D3	142	205	25.3	25.9
DD1	132	238	25.8	26.5
DD2	138	232	17.8	22.6
DD3	152	218	25.6	26.3
E1	132	195	26.2	27.3
E2	132	220	26.5	27.4
E3	136	193	24.7	25.7
E4	138	188	23.4	26.0

TABLE II (Continued)

Specimen	Age at Vacuum Saturation Days	Immersion Period Days	Volumetric Water Content: At Vacuum Saturation %	Volumetric Water Content: At Test %
EE1	132	196	26.6	27.5
EE2	138	190	25.9	26.3
EE3	138	215	19.2	25.0
EE4	142	212	26.0	26.4
F1	136	199	23.7	24.8
F2	138	204	25.2	25.6
F3	142	194	24.9	26.6
FF1	132	229	19.3	26.9
FF2	138	225	25.7	26.2
FF3	142	225	24.9	25.3
G1	132	224	26.1	27.4
G2	138	217	18.7	24.5
G3	142	215	25.7	26.1
GG1	132	227	19.3	27.0
GG2	136	224	22.5	25.8
GG3	142	218	25.7	26.2
H1	132	241	21.0	24.8
H2	138	260	19.5	25.8
H3	142	234	23.0	26.5
H4[a]	246	138	25.9	26.0
HH1	132	255	22.2	27.6
HH2	138	264	18.4	24.8
HH3	142	300	26.1	26.5
HH4	142	246	25.8	26.6
I1	132	291	24.0	27.3
I2	136	273	22.2	24.8
I3	142	265	25.3	25.7
I4[a]	246	161	25.2	25.8
II1	132	275	20.4	27.4
II2	132	292	21.5	26.4
II3	136	273	25.6	26.0
II4	136	274	25.0	25.6
II5	142	266	24.2	26.0

TABLE II (Continued)

Specimen	Age at Vacuum Saturation Days	Immersion Period Days	Volumetric Water Content: At Vacuum Saturation %	Volumetric Water Content: At Test %
J1	132	278	26.3	27.3
J2	132	278	23.9	26.9
J3	138	273	17.4	20.4
J4	142	270	24.4	26.3
JJ1	132	283	21.7	27.8
JJ2	136	283	24.5	25.4
JJ3	138	284	25.7	26.3
JJ4	152	269	25.4	26.4
R1	132	173	24.3	26.6
R2	136	217	23.4	24.6
R3	138	167	25.5	26.0
		4 by 8 in. Specimens		
K1	156	295	20.1	23.4
K2	169	282	25.8	26.5
K3	171	286	18.8	26.9
K4	171	277	21.8	26.0
K5	219	229	26.3	27.2
KK1	156	293	16.4	19.7
KK2	160	297	21.6	24.4
KK3	169	280	19.0	26.8
KK4	171	286	24.3	26.9
KK5	171	288	25.9	27.2
KK6	245	204	17.0	18.8
L1	156	294	16.0	18.6
L2	160	290	21.6	26.7
L3	165	292	19.5	23.7
L4	169	282	25.9	27.1
L5	227	224	18.7	20.4
L6	233	218	18.4	27.5
LL1	156	298	16.4	19.5
LL2	160	299	20.0	25.4
LL3	165	294	19.6	26.1
LL4	169	286	25.9	26.5
LL5	171	282	22.2	27.5
LL6	227	226	22.4	25.7
LL7	245	209	17.2	25.1

TABLE II (Continued)

Specimen	Age at Vacuum Saturation Days	Immersion Period Days	Volumetric Water Content: At Vacuum Saturation %	Volumetric Water Content: At Test %
M1	156	300	24.5	26.5
M2	162	295	25.5	26.9
M3	162	294	27.5	28.0
M4	169	287	19.2	26.5
M5	171	285	20.5	27.5
M6	219	237	25.6	26.3
M7	233	223	22.6	26.0
MM1	160	296	18.4	20.4
MM2	162	294	24.4	26.9
MM3	171	285	26.8	28.1
MM4	219	237	26.6	27.1
MM5	233	224	22.7	26.3
N1	162	302	19.6	26.7
N2	162	298	24.0	26.7
N3	227	233	18.4	20.5
N4	233	226	19.6	26.0
N5	233	228	20.4	25.3
N6	245	216	21.9	25.3
NN1	165	298	22.8	27.4
NN2	165	299	19.3	25.4
NN3	219	243	24.7	27.2
NN4	219	244	26.1	26.7
NN5	227	236	24.1	26.7
NN6	245	217	22.4	26.5
S1	156	289	18.2	20.9
S2	160	285	25.9	26.5
S3	169	276	19.2	26.7
S4	227	218	20.7	25.0
S5	245	200	19.2	23.4

[a]Air dried at room temperature for 96 days after oven drying, prior to vacuum saturation.

TABLE III

INFLUENCE OF IMMERSION ON SATURATION

Specimen Size	Mean Volumetric Water Content %	Standard Deviation %	Coefficient of Variation %	Remarks
3" x 6"	23.8	2.5	10.5	At vacuum saturation
3" x 6"	26.0	1.2	4.6	At test
4" x 8"	21.7	3.2	14.7	At vacuum saturation
4" x 8"	25.3	2.5	9.9	At test

absolute value of the degree of saturation of the specimens cannot be accurately determined.

4.2 Fatigue Tests

The average compressive strength of concrete was determined from standard compression tests, on saturated mortar specimens, selected at random. The age, unit weight, volumetric porosity, and strength of the specimens are given in Table IV.

The test results of all the specimens of sizes 3 by 6 in. and 4 by 8 in. subjected to various types of fatigue loading are given in Tables V and VI, respectively. All the specimens were tested to failure and at least three specimens were tested under each loading and test condition. In the case of 4 by 8 in. specimens tests were carried out only for the range of stress of 0.1 f_c' to 0.7 f_c' and at least five specimens were tested under each loading and test condition.

TABLE IV

PROPERTIES OF SATURATED CONCRETE

Specimen	Age at Test, days	Unit Weight at Test, pcf	Volumetric Water Content at Test, %	Compressive Strength, psi
		3 by 6 in. Specimens		
R1	305	136	26.6	2790
R2	347	136	24.6	2980
R3	305	137	26.6	2620
				Mean f'_c = 2800
				SD,s* = 184
		4 by 8 in. Specimens		
S1	445	133	20.9	2920
S2	445	136	26.5	2830
S3	445	138	26.7	2700
S4	445	136	25.0	2600
S5	445	136	23.4	2670
				Mean f'_c = 2740
				SD,s = 134

*Standard Deviation,s

TABLE V

TEST RESULTS OF 3 BY 6 IN. SPECIMENS

Specimen	Load Type	Age at Test, Days	Volumetric Water Content at Test, %	Number of Cycles to Failure, N
A1	1	309	27.4	591
A2	1	309	25.6	886
A3	1	309	25.4	805
				Mean, $\bar{N}$ = 761
				SD, s^a = 152
AA1	1	311	27.8	270
AA2	1	311	27.7	659
AA3	1	311	26.1	996
				Mean, $\bar{N}$ = 642
				SD, s = 364
B1	1	312	27.6	4046
B2	1	424	25.7	4265
B3	1	312	26.2	5476
B4	1	312	25.5	4831
				Mean, $\bar{N}$ = 4655
				SD, s = 640
BB1	1	312	27.1	3145
BB2	1	312	25.2	1528
BB3	1	312	24.6	3743
BB4	1	425	25.5	2499
				Mean, $\bar{N}$ = 2729
				SD, s = 948
C1	1	331	26.9	11472
C2	1	331	26.0	22111
C3	1	332	23.7	34323
				Mean, $\bar{N}$ = 22635
				SD, s = 11434
CC1	1	348	26.2	33185
CC2	1	348	25.9	24516
CC3	1	349	26.6	16043
				Mean, $\bar{N}$ = 24581
				SD, s = 8570

TABLE V (Continued)

Specimen	Load Type	Age at Test, Days	Volumetric Water Contents at Test, %	Number of Cycles to Failure, N
D1	2	347	25.3	560
D2	2	347	26.0	1519
D3	2	347	25.9	531
				Mean, $\bar{N}$ = 870
				SD,s = 562
DD1	2	370	26.5	305
DD2	2	370	22.6	872
DD3	2	370	26.3	355
				Mean, $\bar{N}$ = 511
				SD,s = 314
E1	2	327	27.3	788
E2	2	352	27.4	1493
E3	2	327	25.7	1816
E4	2	326	26.0	2536
				Mean, $\bar{N}$ = 1658
				SD,s = 725
EE1	2	328	27.5	3945
EE2	2	328	26.3	2763
EE3	2	353	25.0	3476
EE4	2	354	26.4	2332
				Mean, $\bar{N}$ = 3129
				SD,s = 720
F1	2	333	24.8	23647
F2	2	342	25.6	26798
F3	2	336	26.6	46643
				Mean, $\bar{N}$ = 32363
				SD,s = 12467
FF1	2	361	26.9	12418
FF2	2	363	26.2	30149[b]
FF3	2	367	25.3	22566
				Mean, $\bar{N}$ = 21711
				SD,s = 8896
G1	3	356	27.4	330
G2	3	355	24.5	668
G3	3	357	26.1	816
				Mean, $\bar{N}$ = 605
				SD,s = 249

TABLE V (Continued)

Specimen	Load Type	Age at Test, Days	Volumetric Water Content at Test, %	Number of Cycles to Failure, N
GG1	3	359	27.0	533
GG2	3	358	25.8	416
GG3	3	360	26.2	518
				Mean, $\bar{N}$ = 489
				SD,s = 64
H1	3	373	24.8	2582
H2	3	396	25.8	4909
H3	3	376	26.5	5919[c]
H4	3	384	26.0	2883
				Mean, $\bar{N}$ = 4073
				SD,s = 1607
HH1	3	387	27.6	643
HH2	3	402	24.8	2139
HH3	3	442	26.5	1219
HH4	3	388	26.6	4190
				Mean, $\bar{N}$ = 2048
				SD,s = 1555
I1	4	423	27.3	9232
I2	4	407	24.8	18845
I3	4	407	25.7	14969
I4	4	407	25.8	16417
				Mean, $\bar{N}$ = 14866
				SD,s = 4082
II1[d]	4	407	27.4	4808
II2	4	424	26.4	10921
II3	4	407	26.0	46167
II4	4	408	25.6	19516
II5	4	408	26.0	17556
				Mean, $\bar{N}$ = 19794
				SD,s = 15843
J1	5	410	27.3	2485
J2	5	410	26.9	6048
J3	5	411	20.4	6899
J4	5	412	26.3	5462
				Mean, $\bar{N}$ = 5224
				SD,s = 1919

TABLE V (Continued)

Specimen	Load Type	Age at Test, Days	Volumetric Water Content at Test, %	Number of Cycles to Failure, N
JJ1	5	415	27.8	5752
JJ2	5	417	25.4	14389
JJ3	5	422	26.3	8533
JJ4	5	421	26.4	9814
				Mean, $\bar{N}$ = 9622
				SD,s = 3602

[a]Standard Deviation,s

[b]Testing interrupted for 4.5 hours at 981 cycles due to technical problems with the controls of the machine.

[c]Loading stopped at 5833 cylces due to mechanical problems and resumed after 26 hours. Only 86 cycles were applied after testing was resumed.

[d]Beginning with this series, specimens were first wrapped around with perforated plastic sheet and then with a moist paper towel, for tests under moist condition.

TABLE VI

TEST RESULTS OF 4 BY 8 IN. SPECIMENS

Specimen	Load Type	Age at Test, Days	Volumetric Water Content at Test, %	Number of Cycles to Failure, N
K1	1	451	23.4	1431
K2	1	451	26.5	433
K3	1	457	26.9	386
K4	1	448	26.0	1684
K5	1	448	27.2	359
				Mean, $\bar{N}$ = 859
				SD, s[a] = 645
KK1	1	449	19.7	1231
KK2	1	457	24.4	904
KK3	1	449	26.8	2254
KK4	1	457	26.9	1114
KK5	1	449	27.2	1329
KK6	1	449	18.8	5363
				Mean, $\bar{N}$ = 2033
				SD, s = 1697
L1	2	450	18.6	2619
L2	2	450	26.7	1519
L3	2	457	23.7	444
L4	2	451	27.1	700
L5	2	451	20.4	682
L6	2	451	27.5	1613
				Mean, $\bar{N}$ = 1263
				SD, s = 818
LL1	2	454	19.5	3771
LL2	2	459	25.4	995
LL3	2	459	26.1	883
LL4	2	455	26.5	557
LL5	2	453	27.5	1254
LL6	2	453	25.7	1200
LL7	2	454	25.1	899
				Mean, $\bar{N}$ = 1366
				SD, s = 1074
M1	4	456	26.5	2652
M2	4	457	26.9	2720
M3	4	456	28.0	1840
M4	4	456	26.5	4164

TABLE VI (Continued)

Specimen	Load Type	Age at Test, Days	Volumetric Water Content at Test, %	Number of Cycles to Failure, N
M5	4	456	27.5	904
M6	4	456	26.3	593
M7	4	456	26.0	10106
				Mean, $\bar{N}$ = 3283
				SD,s = 3241
MM1	4	456	20.4	3359
MM2	4	456	26.9	5808
MM3	4	456	28.1	2275
MM4	4	456	27.1	3718
MM5	4	457	26.3	6677
				Mean, $\bar{N}$ = 4367
				SD,s = 1818
N1	5	464	26.7	6639
N2	5	460	26.7	2070
N3	5	460	20.5	3321
N4	5	459	26.0	3331
N5	5	461	25.3	2449
N6	5	461	25.3	7507
				Mean, $\bar{N}$ = 4220
				SD,s = 2280
NN1	5	463	27.4	655
NN2	5	464	25.4	2305
NN3	5	462	27.2	1260
NN4	5	463	26.7	1430
NN5	5	463	26.7	1546
NN6	5	462	26.5	2587
				Mean, $\bar{N}$ = 1631
				SD,s = 708

[a]Standard Deviation,s

CHAPTER V

ANALYSIS AND DISCUSSION OF RESULTS

5.1 Saturation Tests

Theoretical estimate of the total volume of voids in any concrete can be obtained by calculation from an assumed value of water-cement ratio required for complete hydration of cement and from the solid volumes of the materials used in the mix. By assuming a value of 0.36 for the water-cement ratio required for complete hydration of cement, the percentage of the total volume of voids on an air free basis to the total volume of concrete was theoretically calculated to be 18.2, for the concrete mix used in this study.

The amount of air voids in the concrete was calculated using the gravimetric method as suggested by C 138-75 of ASTM. The unit weight of concrete used in the calculation was obtained from the moist weight of the specimens taken on removal from the moist room just before the start of oven drying process. The average air contents of the mixes used in 3 by 6 in. and 4 by 8 in. specimens were calculated to be 8.6 and 8.0 percent with coefficients of variation of 0.1 and 7.4 percent, respectively. The average air content of the concrete and the coefficients of variation in the case of larger specimens are rather high.

The average volume of all the voids in the concrete may be taken as the sum of the theoretical volume of voids obtained on an air free basis and the air content. Thus the average volume of voids expressed as a fraction of the total volume of the specimen worked out to be 26.8 and 26.2 percent for the 3 by 6 in. and the 4 by 8 in. specimens, respectively. The above values are to be compared to the average values of volumetric water contents of 26 and 25.3 percent, given in Table III, for the two sizes of specimens. Hence the average degrees of saturation at test for the 3 by 6 in. and 4 by 8 in. specimens worked out to 97 and 96.6 percent, respectively. Though the average degree of saturation was very high, there were a few specimens with degree of saturation as low as 75 percent. However, the fatigue life of such specimens was not found to be markedly different from the rest of the specimens in the group for a given test series.

5.2 Statistical Analysis

Statistical analysis is an indispensable tool in the analysis of fatigue data which usually tends to display a wide scatter. It usually happens that unit observations to be analyzed can be classified into two or more groups. In this study, the groups of data are on the number of cycles to failure. As would be expected, a wide scatter usually exists in the data on number of cycles to failure for similar specimens tested under similar conditions. Several groups of data on the number of cycles to failure have been obtained for a number of variations in test conditions. Though the average number of cycles to failure can be obtained separately for each group of data, there exists a reasonable question if the difference exhibited in average values of the groups is statistically significant. If there is no statistical difference between the groups, they could be considered as one large group. The same is true of the statistical difference, if any, between regression lines obtained for groups of fatigue data. In this study, both these problems have been approached by use of a covariance analysis.

While comparing two groups of data on the number of cycles to failure, the actual cycles as well as the logarithm of the number of cycles of the groups were compared. Since it is usual practice to present the fatigue results in terms of semi-logarithmic plots, it may be adequate to compare logarithm of the number of cycles of two groups for statistical significance. All statistical tests were made at the 5 percent significance level.

5.3 Effect of Submergence

High pore pressures are expected to develop during axial compressive loading of saturated concrete. These pore pressures could initiate microfracturing of concrete and cause increase in pore volume and thus cause volumetric dilation. Replenishing the pore water squeezed out during the loading phase and the filling up of the additional pore space, due to fracturing, with water depends on several factors. The factors which may be expected to have considerable influence are, the availability of a recharge source, the distance of the recharge source from the center of the specimen, the pressure of the recharge source, the rest periods between the application of load peaks, and the intrinsic pore characteristics of the specimen such as pore size and continuity. In addition to studying the influence of a few other variables, it is proposed to study in greater detail the influence of a recharge source and rest periods.

To study the influence of the presence of a recharge source to the pore water, two test series were run for each load type and range of stress and the results compared. In the first series the saturated specimens were kept submerged in water during the tests. Thus the maximum distance to the recharge source was the radius of the specimen. In the second series, the saturated specimens were merely kept moist during tests, which merely prevented the loss of pore water to evaporation but no recharge source was available to the pore water.

In Table VII the average number of cycles to failure of submerged saturated specimens subjected to various types of loadings are compared with those of their companion specimens tested in a moist condition. A study of the results presented shows that of the 14 pairs of test data, only three pairs of data obtained for the submerged and moist conditions are statistically different. The average number of cycles to failure for submerged specimens was higher in two cases when compared with those of their companion moist specimens. In the third case, the moist specimens recorded higher number of cycles to failure. The three cases were subjected to three different load types but were subjected to approximately the same stress range.

Thus, the number of observed differences in fatigue strength of saturated concrete tested under submerged and moist conditions is limited and neither the observed differences were confined to a single load type nor followed a single trend. The observed differences in fatigue life of saturated concrete tested under submerged and moist conditions do not appear to be linked to either load type or the rate of loading in a definite pattern. One of the reasons for the absence of any perceptible difference in behavior when tested under submerged or moist condition may lie in the fact that the specimens contained a large number of entrapped air voids ranging in diameter from approximately 0.5 to 1 mm. These large

TABLE VII

INFLUENCE OF SUBMERGENCE

Max. Stress f'_c	Period of Load Per Cycle, Sec.	Rest Period Per Cycle, Sec.	Mean No. of Cycles to Failure: Submerged	Mean No. of Cycles to Failure: Moist
		3 by 6 in.; Minimum Stress Level at 0.096 f'_c		
0.77	1.0	0.0	761 (A)*	642 (AA)*
0.67	1.0	0.0	4655 (B)	2729[a] (BB)
0.57	1.0	0.0	22635 (C)	24581 (CC)
0.77	1.0	9.0	870 (D)	511 (DD)
0.67	1.0	9.0	1658 (E)	3129[a] (EE)
0.57	1.0	9.0	32363 (F)	21711 (FF)
0.77	1.0	99.0	605 (G)	489 (GG)
0.67	1.0	99.0	4073 (H)	2048 (HH)
0.67	0.1	0.0	14866 (I)	19794 (II)
0.67	0.1	9.9	5224 (J)	9622 (JJ)
		4 by 8 in.; Minimum Stress Level at 0.1 f'_c		
0.70	1.0	0.0	859 (K)	2033 (KK)
0.70	1.0	9.0	1263 (L)	1366 (LL)
0.70	0.1	0.0	3283 (M)	4367 (MM)
0.70	0.1	9.9	4220 (N)	1631[a] (NN)

[a]The value is statistically different from the corresponding value for the submerged condition.

*Test Series.

air voids when completely filled with water may have acted as mini-reservoirs with sufficient capacity to replenish the pore water in the specimens tested under moist condition.

5.4 Range of Loading

The three different ratios of minimum stress level to maximum stress level, R, used in this study were 0.125, 0.143, and 0.167. The average number of cycles to failure for each range of loading and condition of tests may be seen in Table VII. With increasing R values, that is with reduced range of loading, and with decreasing maximum stress level, the number of cycles showed an increase in fatigue strength in accordance with the idea expressed in the modified Goodman diagram on fatigue strength of concrete. Figures 9 and 10 show the S-N diagrams obtained from the results of 3 by 6 in. specimens subjected to axial compressive loading without rest periods and tested under submerged and moist conditions, respectively. The results of a linear regression of stress level versus logarithm of cycles to failure for the specimens is also shown. For comparison, the regression lines obtained for the submerged tests and the moist tests are plotted in Figure 11. It may be seen that there is practically no difference between the regression lines obtained for the two conditions of testing. Also statistical comparison of the two groups of data indicated neither a significant difference in slope nor a significant difference in level between the two groups. Hence, a combined regression line obtained from the data on moist and submerged condition tests is also shown in the figure. This may be considered as the general fatigue curve obtained for saturated concrete subjected to axial compressive loading. Figure 11 also shows the fatigue curve obtained by Antrim

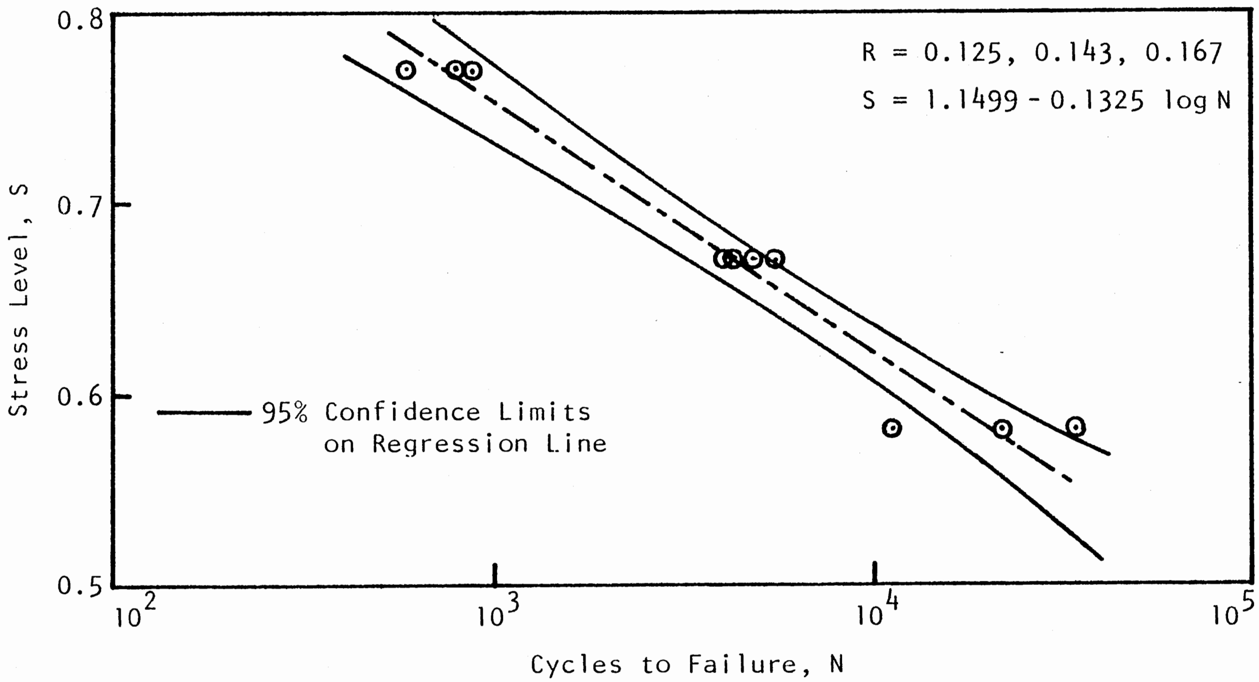


Figure 9. S-N Diagram for Saturated Concrete Subjected to Repeated Axial Compressive Loading (Submerged Condition)

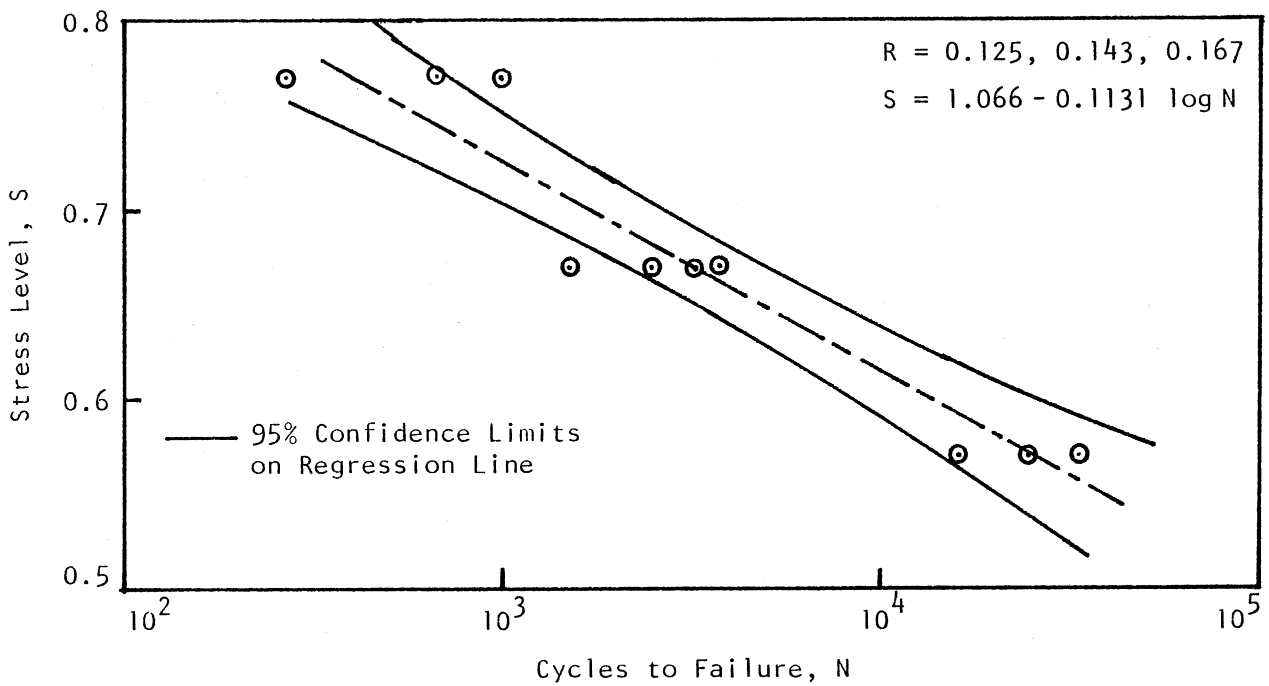


Figure 10. S-N Diagram for Saturated Concrete Subjected to Repeated Axial Compression Loading (Moist Condition)

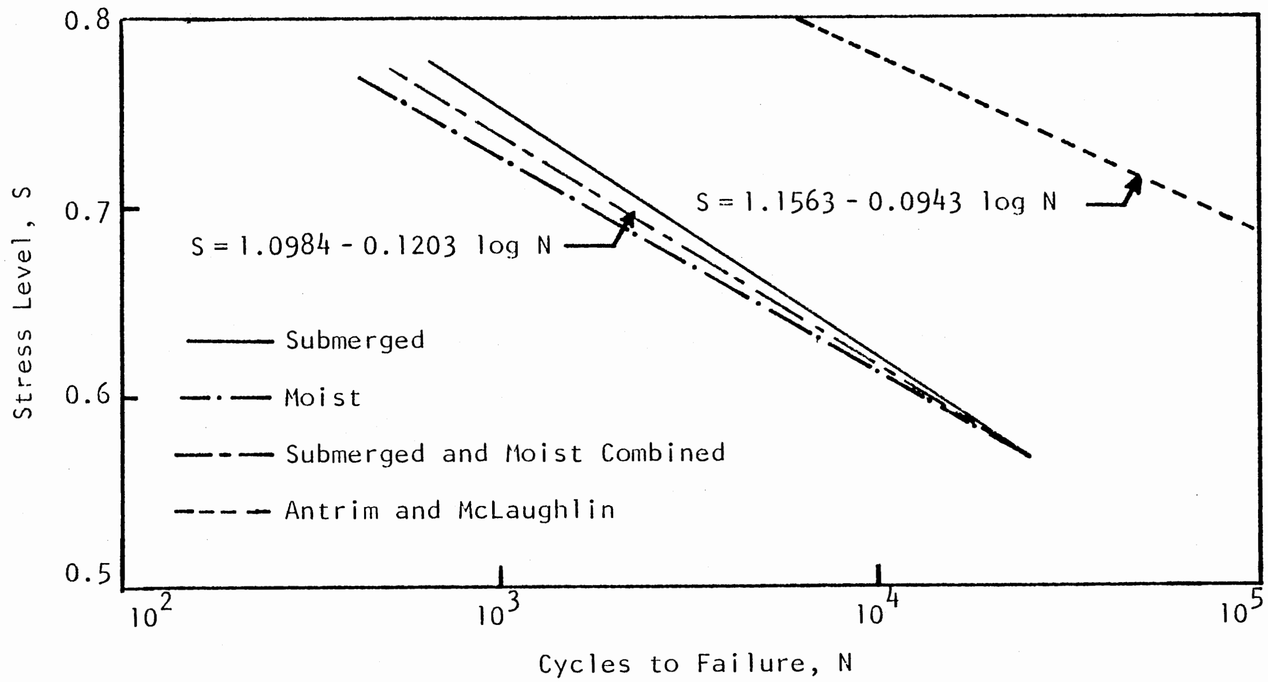


Figure 11. Fatigue Curves for Saturated Concrete Subjected to Repeated Axial Compressive Loading

and McLaughlin (56) from their tests on non-air entrained concrete subjected to axial fatigue loading. It may be mentioned that the frequency of loading used in their study was 16.7 Hz as compared to 1 Hz used in the present study. The size of the specimens used in their study was 3 by 6 in. and they were oven dried for 3 to 4 days prior to testing. The range of stress, R, used in their study was very close to zero and the maximum stress level ranged from 66 to 84 percent of f'_c. Thus all the conditions of testing, except for range of stress, saturation of the specimens and moisture condition during testing, were similar and comparable to the ones used in the present study and thus provides a valid comparison of the fatigue strengths of saturated and dry concrete. From a comparison it may be seen that the ratio of the fatigue strength of saturated concrete to the fatigue strength of dry concrete at any given number of cycles is approximately 0.75. The lower strength of saturated concrete is to be expected in view of the possible fracturing of the concrete caused by the load induced pore pressures. In order to compare the fatigue strengths of saturated and dry concrete subjected to comparable loadings, the data from test series involving rest periods and higher rates of loading some of which formed a statistically different group were not included in this section.

5.5 Effect of Rest Periods

In Table VIII the average number of cycles to failure of saturated concrete obtained for loadings with and without rest periods are compared. In most cases the rest period was either 9.0 or 99.0 seconds. However, for loadings at the frequency of 10 Hz the rest period was 9.9 seconds. Only in three cases does the introduction of rest periods in

TABLE VIII

INFLUENCE OF REST PERIODS

Max. Stress / f'_c	Moisture Condition	Period of Load Per Cycle, Sec.	Mean No. of Cycles to Failure: Rest Period, 0 Sec.	Rest Period, 9 Sec.	Rest Period, 99 Sec.
		3 by 6 in. Specimens; Minimum Stress Level, 0.096 f'_c			
0.77	Submerged	1.0	761 (A)	870 (D)	605 (G)
0.77	Moist	1.0	642 (AA)	511 (DD)	489 (GG)
0.67	Submerged	1.0	4655 (B)	1658[b] (E)	4073[c] (H)
0.67	Moist	1.0	2729 (BB)	3129 (EE)	2048 (HH)
0.57	Submerged	1.0	22635 (C)	32363 (F)	---
0.57	Moist	1.0	24581 (CC)	21711 (FF)	---
0.67	Submerged	0.1	14866 (I)	5224[a,b] (J)	---
0.67	Moist	0.1	19794 (II)	9622[a] (JJ)	---
		4 by 8 in. Specimens; Minimum Stress Level, 0.1 f'_c			
0.70	Submerged	1.0	859 (K)	1263 (L)	---
0.70	Moist	1.0	2033 (KK)	1366 (LL)	---
0.70	Submerged	0.1	3283 (M)	4220[a] (N)	---
0.70	Moist	0.1	4367 (MM)	1631[a,b] (NN)	---

[a]Rest period is 9.9 seconds.

[b]The value is statistically different from the corresponding value obtained for the rest period of 0 seconds.

[c]The value is statistically different from the corresponding value obtained for the rest period of 9 seconds but is not statistically different from the value obtained for the rest period of 0 seconds.

the loading spectrum yield data that are statistically different from that of the specimens subjected to loading without rest periods. In two cases interspersing of rest periods in the loading sequence is found to be detrimental and both these cases are associated with a higher rate of loading. The S-N diagrams for the specimens subjected to loading with rest periods under submerged and moist conditions are presented in Figures 12 and 13. The scatter of the data, particularly at the stress level of 0.67 is great. There is no statistically significant difference in the levels or slope of data obtained for the submerged and moist conditions. Hence the two groups are combined and the fatigue curves of saturated concrete subjected to axial compressive loading with rest periods introduced is presented in Figure 14. It is seen that the introduction of rest periods has practically no effect on the fatigue strength of saturated concrete. This is contrary to the increase in fatigue strengths in flexure for plain concrete beams observed by Hilsdorf and Kesler (47) for tests with rest periods between one and five minutes. The introduction of rest periods was expected to give lower fatigue strength for saturated concrete tested submerged because of the recharge of pore water made possible during the rest period. However, the rest periods 9.0 and 99.0 seconds used in this study may not have been sufficient for recharging the pore water considering the enormous time taken by the specimens to achieve saturation. It is also probable that the benefits of introducing rest periods were neutralized by the detrimental effect of the pore pressures developed in the saturated specimen.

5.6 Rate of Loading

Specimens used in this study were subjected to two different rates

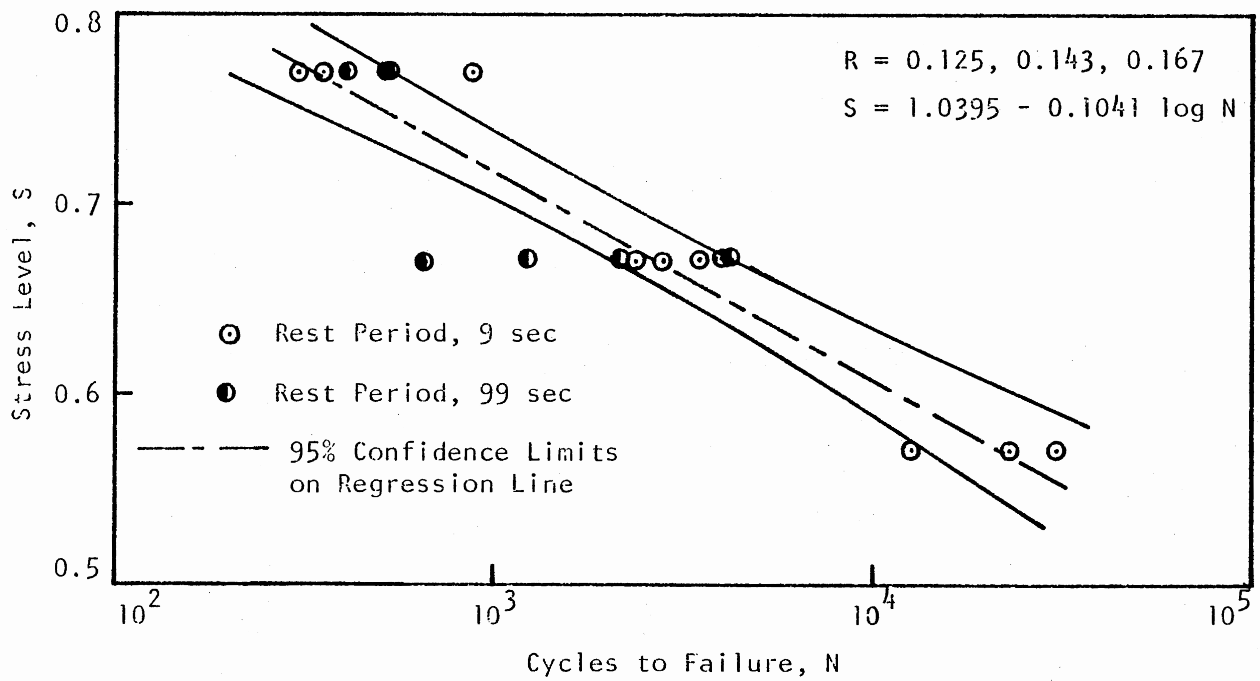


Figure 12. S-N Diagram for Saturated Concrete Subjected to Repeated Axial Compressive Loading With Rest Periods (Submerged Condition)

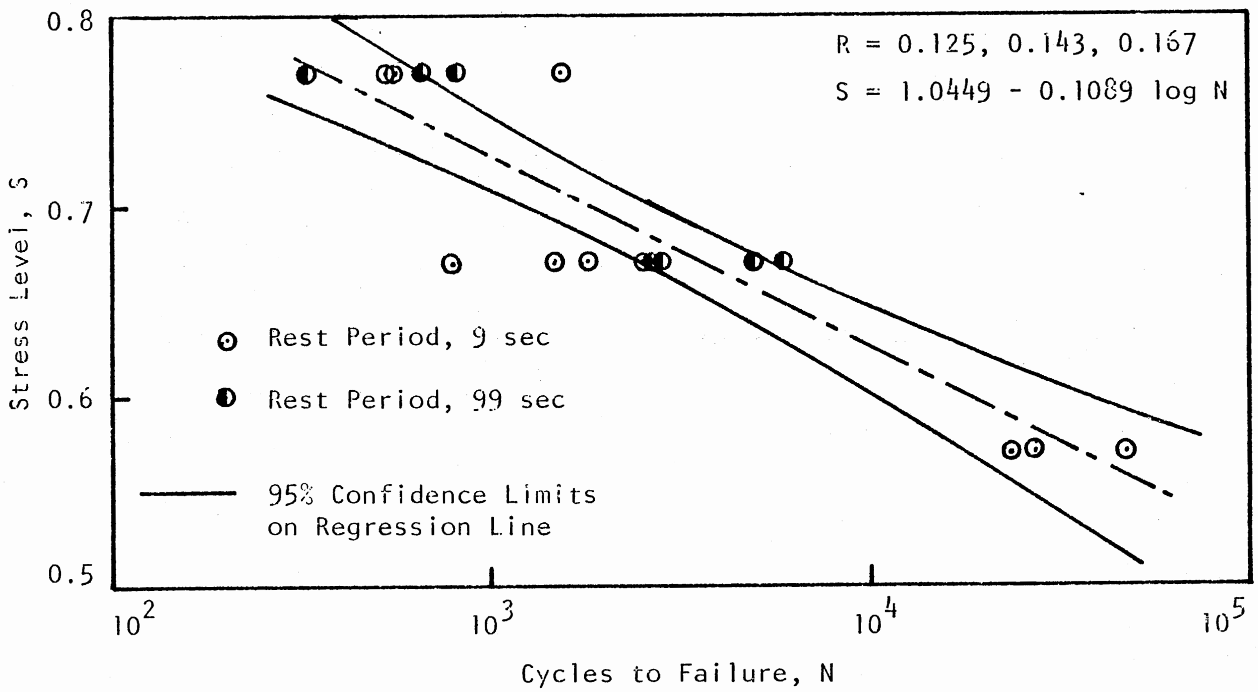


Figure 13. S-N Diagram for Saturated Concrete Subjected to Repeated Axial Compressive Loading With Rest Periods (Moist Condition)

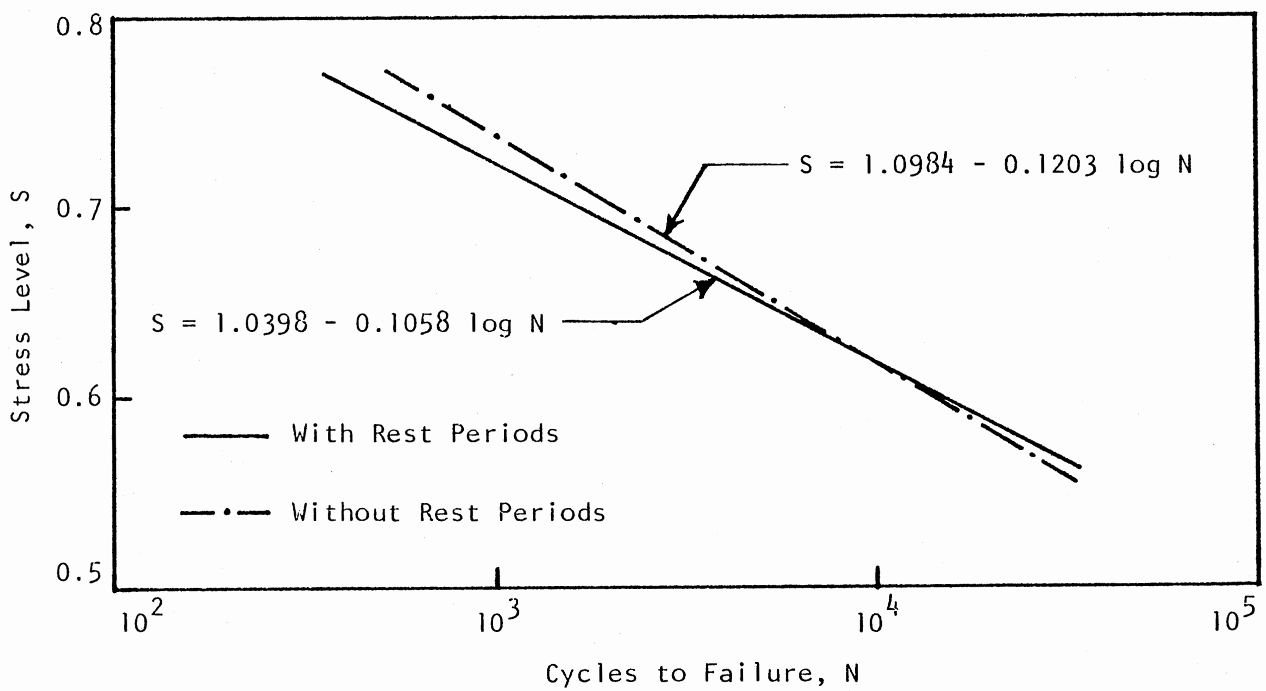


Figure 14. Fatigue Curves for Saturated Concrete Subjected to Repeated Axial Compressive Loading With Rest Periods

of loading, 1 and 10 Hertz, and the results compared for tests in which the other parameters like the moisture condition and range of loading were the same. Table IX shows the average number of cycles to failure obtained for the various tests at frequencies of 1 and 10 Hertz. From the results of 3 by 6 in. specimens, it is seen that the average number of cycles to failure obtained at higher frequency was always higher than that obtained for the lower rate of loading. The groups of data obtained for the two rates of loading are also found to be statistically different. For the increase in rate of loading from 1 to 10 Hertz, the average number of cycles to failure was found to increase substantially for both sizes of specimens. The amount of increase observed must be viewed with caution as the absolute number of cycles to failure obtained for the fatigue tests is very low when compared with the results for dry concrete reported in literature. Kesler (57) from his studies had found that frequencies of loading from 1.1 to 7.3 Hertz had negligible influence on the fatigue strength of concrete. However, studies of Sparks and Menzier (58) indicated a tenfold increase in the fatigue life of concrete for a hundredfold increase in the rate of loading.

5.7 Influence of Size

The average number of cycles to failure obtained for 3 by 6 in. and 4 by 8 in. specimens subjected to the same type of loading and similar test conditions are compared in Table X. Of the eight pairs of test series compared statistical difference between the groups of data was found to exist only in five cases. The average number of cycles to failure for the larger specimens was always lower than that obtained for the smaller specimens. The reduction in the average number of cycles to

TABLE IX

INFLUENCE OF RATE OF LOADING

Rest Period Sec.	Moisture Condition	Mean No. of Cycles to Failure: Load Period 1.0 Sec.	Mean No. of Cycles to Failure: Load Period 0.1 Sec.
	3 by 6 in. Specimens; Max. Stress Level, 0.67 f'_c; Min. Stress Level, 0.096 f'_c		
0.0	Submerged	4655 (B)	14866[b] (I)
0.0	Moist	2729 (BB)	19794[c] (II)
9.0	Submerged	1658 (E)	5224[a,b] (J)
9.0	Moist	3129 (EE)	9622[a,b] (JJ)
	4 by 8 in. Specimens; Max. Stress Level, 0.7 f'_c; Min. Stress Level, 0.1 f'_c		
0.0	Submerged	859 (K)	3283[c] (M)
0.0	Moist	2033 (KK)	4367[c] (MM)
9.0	Submerged	1263 (L)	4220[a,b] (N)
9.0	Moist	1366 (LL)	1631[a] (NN)

[a]Rest period is 9.9 seconds.

[b]The value is statistically different from the corresponding value obtained for the load period of 1.0 second.

[c]The value is statistically different from the corresponding value obtained for the load period of 1.0 sec. only if logarithm of the number of cycles to failure are compared in the statistical analysis.

failure with increase in size of the specimens was approximately found to be equal to 75 percent.

TABLE X

INFLUENCE OF SIZE

Load Type	Min. Stress / Max. Stress	Mean No. of Cycles to Failure: 3 by 6 in.		Mean No. of Cycles to Failure: 4 by 8 in.		Moisture Condition
1	0.143	4655	(B)	859[a]	(K)	Submerged
1	0.143	2729	(BB)	2033	(KK)	Moist
2	0.143	1658	(E)	1263	(L)	Submerged
2	0.143	3129	(EE)	1366[a]	(LL)	Moist
4	0.143	14866	(I)	3283[a]	(M)	Submerged
4	0.143	19794	(II)	4367[b]	(MM)	Moist
5	0.143	5224	(J)	4220	(N)	Submerged
5	0.143	9622	(JJ)	1631[a]	(NN)	Moist

[a]The value is statistically different from the corresponding value obtained for the 3 by 6 in. specimen.

[b]The value is statistically different from the corresponding value obtained for the 3 by 6 in. specimen only if the logarithm of the number of cycles to failure are compared in the statistical analysis.

CHAPTER VI

SUMMARY AND CONCLUSIONS

6.1 Summary

The purpose of this study was twofold. Firstly, to critically review the current state and limitations of the knowledge on the various aspects of pore pressure in concrete as compared to the store of information available on pore pressure effects in the area of soil and rock mechanics; secondly, to attempt to saturate concrete using a vacuum saturation technique and to conduct fatigue tests on saturated concrete to obtain information on the possible effects of pore pressure as reflected in the fatigue characteristics of saturated concrete.

Specimens used in this study consisted of cement mortar cylinders of sizes 3 by 6 in. and 4 by 8 in. Prior to fatigue testing all the specimens were oven dried, evacuated for removal of the entrapped air, and then saturated using deaired water. In all, 71 specimens of 3 by 6 in. and 48 specimens of 4 by 8 in. were subjected to detailed fatigue testing program. The minimum stress level in all the fatigue tests was approximately 0.1 f'_c, f'_c being obtained from static strength tests on similarly saturated specimens of the same size. The ratios of the minimum to the maximum stress levels were 0.125, 0.143, and 0.167. Some specimens were also tested using load spectra that contained rest periods of 9.0, 9.9, and 99.0 seconds between each load application. Approximately one-half of the specimens were tested under water and the other half tested under moist condition.

6.2 Conclusions

Even dense concrete is inherently porous and even though the volumetric porosity may be low, the boundary porosity of concrete is very close to one. Saturated rocks as well as saturated concrete have been found to develop sufficient pore pressures and obey the effective stress law. Saturated rocks subjected to rapid loading rates have been observed to develop negative pore pressures due to volumetric dilation and consequent increase in strength. No experimental evidence is available on the aspect of dilatancy hardening or on the general effects of pore pressure on the mechanical properties of concrete.

From the results of fatigue tests on saturated concrete specimens, it may be concluded that:

1. No significant difference is observed in the fatigue strengths of concrete tested under submerged and moist conditions.

2. An increase in the fatigue strength occurs with a decrease in range of loading applied. However, the number of cycles to failure obtained for the saturated concrete is lower by one or two orders of magnitude when compared with that of unsaturated concrete depending on the stress level at which the values are compared.

3. Rest periods of 9.0, 9.9, and 99.0 seconds interspersed in the loading spectra neither increased nor decreased the number of cycles to failure for a given range of loading.

4. Rate of loading was found to have a pronounced effect on the number of cycles to failure at a given stress level. With increase in

the rate of loading, increase in the number of cycles to failure was recorded. The percentage increase was greater for the smaller specimens than for the larger specimens.

5. For any given type of loading and range of stress, the larger specimens recorded substantially lower number of cycles to failure when compared to the smaller specimens.

6.3 Suggestions for Future Work

Prior to this investigation, there was no information available on the fatigue characteristics of saturated concrete. The present study was exploratory and not intended to completely define all aspects of the problem associated with the behavior and response of saturated concrete subjected to fatigue loading. The results from this study indicated that the fatigue strength of saturated concrete is much lower than that of normal dry concrete. The oven drying and evacuation employed for saturation of the specimens may have influenced the results. Hence future work should employ various means of achieving saturation of concrete. A group of specimens containing entrained and entrapped air voids and not subjected to a saturation procedure should be retained as a control group and their results compared with results from the group of specimen subjected to saturation procedure.

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APPENDIX

GRAVIMETRIC DATA FOR SPECIMENS

TABLE XI

GRAVIMETRIC DATA FOR SPECIMENS

Specimen	Dimensions Mean Dia., in.	Mean Height, in.	Moist wt., g.	Oven Dried wt., g.	Vac. Sat. wt., g.	Imm. Sat. wt., g.
			3 by 6 in. Specimens			
A1	3.02	5.82	1448.1	1320.0	1550.8	1576.1
A2	3.03	5.82	1446.1	1329.6	1586.3	1590.5
A3	3.03	5.78	1444.4	1331.2	1566.2	1571.1
AA1	3.03	5.77	1430.2	1304.3	1574.6	1585.5
AA2	3.03	5.72	1424.7	1300.4	1567.5	1573.1
AA3	3.03	5.81	1441.8	1321.9	1562.4	1567.7
B1	3.03	5.78	1435.5	1309.1	1561.5	1575.5
B2	3.06	5.98	1479.1	1358.5	1618.1	1624.6
B3	3.03	5.78	1440.5	1323.1	1564.5	1579.1
B4	3.04	5.78	1437.9	1321.1	1564.2	1566.8
BB1	3.03	5.80	1448.2	1320.6	1565.9	1586.2
BB2	3.04	5.79	1449.6	1330.5	1588.1	1605.5
BB3	3.04	5.79	1436.9	1315.2	1515.2	1558.6
BB4	3.03	5.80	1440.9	1318.7	1563.5	1564.2
C1	3.04	5.71	1426.7	1301.7	1564.1	1569.5
C2	3.04	5.88	1458.5	1342.8	1594.9	1599.2
C3	3.04	5.80	1432.4	1318.7	1528.0	1551.9
CC1	3.03	5.71	1420.9	1297.0	1526.8	1559.2
CC2	3.04	5.87	1459.8	1340.9	1590.0	1592.7
CC3	3.03	5.74	1426.0	1306.2	1562.1	1565.6
D1	3.03	5.75	1435.4	1321.6	1535.7	1574.7
D2	3.04	5.64	1397.8	1287.4	1550.2	1552.8
D3	3.04	5.87	1460.2	1343.4	1592.1	1596.3
DD1	3.02	5.84	1471.1	1341.8	1594.7	1599.5
DD2	3.04	5.87	1469.3	1347.9	1550.8	1584.0
DD3	3.04	5.71	1428.9	1308.2	1559.7	1564.7
E1	3.03	5.80	1450.1	1322.0	1580.3	1587.8
E2	3.03	5.72	1422.7	1295.1	1563.7	1570.0
E3	3.03	5.87	1467.5	1348.3	1598.1	1604.6
E4	3.03	5.77	1425.0	1308.4	1545.0	1563.1
EE1	3.03	5.67	1404.4	1283.6	1552.9	1559.2
EE2	3.04	5.71	1413.3	1300.3	1555.6	1558.5
EE3	3.03	5.81	1439.7	1320.0	1525.9	1565.9
EE4	3.04	5.73	1423.2	1307.0	1561.4	1563.6

TABLE XI (Continued)

Specimen	Dimensions Mean Dia., in.	Dimensions Mean Height, in.	Moist wt., g.	Oven Dried wt., g.	Vac. Sat. wt., g.	Imm. Sat. wt., g.
F1	3.04	5.85	1460.7	1343.0	1581.8	1589.4
F2	3.03	5.80	1446.8	1329.4	1575.0	1577.6
F3	3.04	5.68	1397.2	1286.0	1544.9	1556.2
FF1	3.04	5.78	1441.1	1314.0	1517.7	1569.4
FF2	3.03	5.78	1434.1	1316.3	1560.6	1564.1
FF3	3.04	5.79	1445.4	1329.8	1567.1	1569.8
G1	3.03	5.83	1447.9	1322.0	1572.9	1581.5
G2	3.04	5.77	1431.6	1311.3	1511.9	1551.2
G3	3.04	5.71	1412.3	1300.5	1551.7	1554.6
GG1	3.04	5.70	1412.5	1289.5	1511.9	1564.6
GG2	3.04	5.82	1445.9	1327.2	1561.9	1584.6
GG3	3.03	5.79	1436.8	1321.1	1554.1	1557.6
H1	3.03	5.81	1438.2	1312.3	1536.9	1563.1
H2	3.03	5.79	1438.1	1319.7	1526.4	1569.6
H3	3.03	5.77	1429.9	1314.6	1538.8	1563.0
H4	3.03	5.76	1436.2	1314.0	1570.1	1571.1
HH1	3.03	5.79	1435.7	1310.2	1553.6	1590.4
HH2	3.04	5.74	1429.0	1309.4	1517.7	1561.2
HH3	3.04	5.78	1434.8	1317.1	1566.4	1568.9
HH4	3.03	5.86	1446.3	1330.3	1571.2	1577.0
I1	3.04	5.71	1418.1	1294.4	1554.1	1576.1
I2	3.03	5.71	1424.6	1309.1	1537.1	1555.1
I3	3.03	5.69	1419.8	1308.7	1558.8	1561.8
I4	3.04	5.69	1406.0	1287.0	1534.9	1539.0
II1	3.03	5.76	1433.8	1307.7	1536.4	1584.1
II2	3.03	5.82	1442.3	1316.2	1552.1	1586.0
II3	3.04	5.78	1433.0	1319.2	1565.3	1567.9
II4	3.04	5.71	1424.1	1308.8	1556.4	1560.4
II5	3.03	5.79	1434.3	1320.4	1548.7	1560.9
J1	3.03	5.72	1425.1	1300.3	1551.9	1558.7
J2	3.04	5.77	1428.9	1304.6	1529.8	1550.3
J3	3.03	5.74	1428.2	1309.1	1516.7	1536.9
J4	3.04	5.71	1428.1	1310.9	1546.3	1559.5
JJ1	3.03	5.73	1423.3	1299.0	1533.4	1574.6
JJ2	3.04	5.82	1446.1	1329.3	1567.6	1574.3
JJ3	3.04	5.75	1431.1	1311.6	1559.1	1562.8
JJ4	3.04	5.88	1464.5	1340.5	1612.6	1620.0

TABLE XI (Continued)

Specimen	Dimensions Mean Dia., in.	Mean Height, in.	Moist wt., g.	Oven Dried wt., g.	Vac. Sat. wt., g.	Imm. Sat. wt., g.
R1	3.04	5.84	1452.3	1326.1	1571.7	1587.7
R2	3.04	5.77	1442.7	1328.3	1561.4	1569.9
R3	3.03	5.79	1443.2	1323.9	1566.0	1569.2
4 by 8 in. Specimens						
K1	3.99	7.87	3387.8	3102.2	3583.8	3637.8
K2	3.99	7.89	3444.0	3149.4	3719.6	3731.7
K3	3.98	7.87	3398.7	3110.6	3538.6	3668.0
K4	3.98	7.88	3362.1	3090.2	3578.0	3645.9
K5	3.99	7.89	3405.0	3112.0	3679.4	3693.2
KK1	3.99	7.91	3414.8	3134.4	3548.0	3601.6
KK2	3.99	7.90	3414.9	3129.1	3626.5	3672.0
KK3	3.97	7.90	3397.2	3105.6	3542.8	3667.6
KK4	3.99	7.84	3388.5	3115.7	3652.1	3694.6
KK5	3.99	7.87	3388.3	3095.8	3688.1	3709.2
KK6	3.98	7.90	3398.7	3127.5	3544.2	3572.3
L1	3.98	7.88	3389.0	3110.2	3525.2	3566.8
L2	3.99	7.93	3409.6	3128.6	3638.1	3719.8
L3	3.99	7.86	3417.7	3129.5	3588.3	3656.0
L4	3.98	7.89	3407.0	3114.7	3654.6	3674.5
L5	3.97	7.95	3379.9	3096.9	3572.0	3600.2
L6	3.98	7.83	3368.1	3088.8	3511.5	3656.0
LL1	3.99	7.89	3373.1	3092.4	3518.8	3569.1
LL2	4.00	7.99	3443.7	3156.7	3652.7	3740.1
LL3	3.98	7.94	3432.6	3145.3	3593.6	3698.7
LL4	3.99	7.86	3409.7	3115.8	3649.9	3660.0
LL5	3.99	7.87	3403.3	3112.3	3587.5	3672.3
LL6	3.97	7.91	3377.1	3077.6	3597.4	3651.3
LL7	3.99	7.63	3289.6	3023.8	3442.0	3564.5
M1	3.97	7.89	3402.0	3113.7	3650.4	3682.1
M2	3.99	7.92	3432.0	3136.8	3686.9	3710.0
M3	3.99	7.86	3409.2	3112.8	3687.5	3695.5
M4	3.99	7.93	3417.1	3125.6	3578.4	3697.4
M5	3.98	7.91	3420.7	3124.8	3588.5	3701.1
M6	3.98	7.89	3449.4	3153.9	3749.3	3760.0
M7	3.98	7.88	3416.4	3126.8	3653.5	3707.8
MM1	3.99	7.85	3364.3	3079.7	3580.0	3612.6
MM2	3.98	7.89	3432.0	3133.1	3657.5	3698.0
MM3	3.98	7.82	3410.6	3124.8	3684.4	3705.5

TABLE XI (Continued)

Specimen	Dimensions Mean Dia., in.	Mean Height, in.	Moist wt., g.	Oven Dried wt., g.	Vac. Sat. wt., g.	Imm. Sat. wt., g.
MM4	3.98	7.87	3411.3	3109.4	3687.1	3694.4
MM5	3.98	7.84	3403.4	3121.8	3644.7	3702.0
N1	3.98	7.86	3376.0	3098.4	3547.6	3661.6
N2	3.98	7.86	3425.0	3126.3	3655.6	3699.2
N3	3.97	7.90	3389.7	3099.1	3537.3	3570.5
N4	3.99	7.87	3386.7	3111.4	3555.6	3658.9
N5	3.99	7.87	3384.4	3107.1	3589.7	3668.0
N6	3.98	7.65	3310.2	3031.3	3534.1	3586.6
NN1	3.98	7.79	3363.9	3073.6	3590.3	3663.0
NN2	3.98	7.91	3403.3	3116.1	3571.5	3669.0
NN3	3.99	7.90	3400.8	3110.2	3661.8	3702.1
NN4	3.99	7.83	3398.0	3101.4	3676.9	3686.3
NN5	3.97	7.98	3413.8	3116.9	3666.0	3707.7
NN6	3.98	7.79	3360.2	3077.7	3590.9	3655.1
S1	3.98	7.86	3359.2	3079.6	3530.5	3574.6
S2	3.99	7.94	3379.6	3098.7	3669.8	3679.4
S3	3.98	7.90	3416.1	3121.5	3599.2	3719.2
S4	3.96	7.91	3378.5	3082.6	3579.0	3646.9
S5	3.99	7.88	3423.2	3134.0	3586.6	3655.0

VITA 2

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